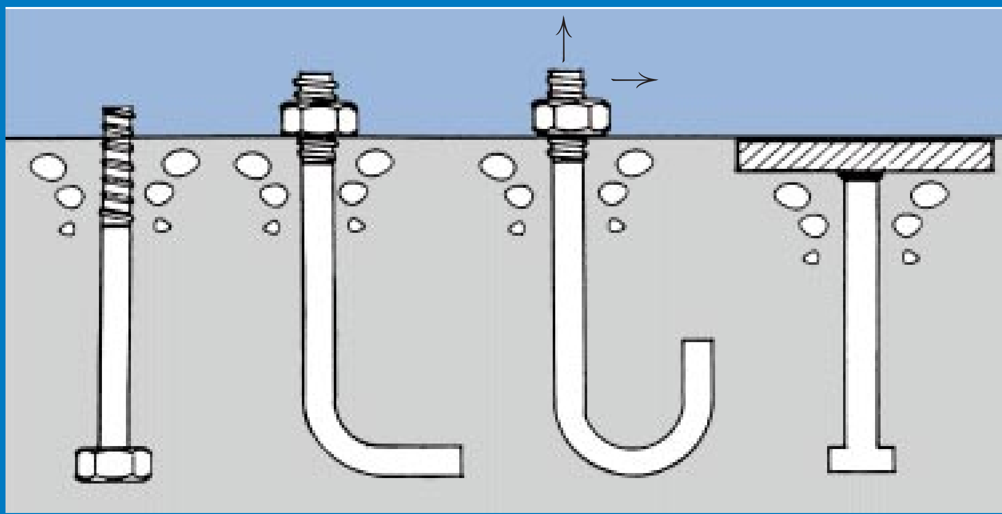
Strength Design of Anchorage to Concrete

By Ronald A. Cook



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Book Contents

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by Ronald A. Cook

PORTLAND CEMENT ASSOCIATION

An organization of cement manufacturers to improve and extend the uses of portland cement and concrete through market development, engineering, research, education, and public affairs work.

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Preface

During the ACI 318-89 and ACI 318-95 code seminars, engineers and architects asked repeatedly about when the ACI 318 Building Code would include design provisions for anchorage to concrete. For the last several years, ACI Committees 318 (Standard Building Code), 349 (Concrete Nuclear Sructures), and 355 (Anchorage to Concrete) have worked diligently on developing such provisions. As discussed in the Introduction of this publication, Committee 318 came very close to completing an Appendix D to ACI 318-99, with code provisions and commentary, for strength design of anchorage to concrete. ACI 318 Appendix D continues to be processed under the ACI consensus process. The code provisions and commentary of the proposed ACI 318 Appendix D for cast-in-place anchors only are reproduced in Appendices A and B of this document, respectively. The contents of Appendices A and B are copyrighted material and reproduced with permission from the American Concrete Institute (ACI). These two appendices are not an official ACI document, but rather part of a document being processed as an ACI consensus standard. The document, as well as the final consensus standard, will be the copyright of ACI. With permission from ACI, the design provisions of Appendix A of this publication are incorporated, with limited amendments, into the first edition of the International Building Code (IBC 2000) as Section 1913.

The primary purpose for publishing this document is to assist the engineer and architect in the implementation of the design provisions for anchorage to concrete. Six design examples illustrate the application of the design provisions for single and group anchors; subject to tension, shear, or combined tension and shear; with or without eccentricity; and with or without edge effects.

PCA would be grateful to any reader who would bring to our attention any errors and inconsistencies found in this first edition. Other suggestions for improvement are also most sincerely welcome.

Basile G. Rabbat, Manager
Transportation Structures and Structural Codes

Acknowledgments

The development of building code provisions for the design of anchorage to concrete has been the result of untold work by many individuals and organizations over the past 25 years. The primary organizations involved have been the American Concrete Institute (ACI) and the Prestressed Concrete Institute (PCI). Early and continuing work in this area has involved ACI Committee 349, Concrete Nuclear Structures, ACI Committee 355, Anchorage to Concrete, and the PCI Industry Handbook Committee. More recently, ACI Committee 318 and in particular Subcommittee B of ACI 318 have worked to produce the design procedures used in this publication. Some of the key individuals involved over the past 25 years include: Mr. Robert W. Cannon as the primary author of the 45-deg cone method first published by ACI 349 in 1976; Dr. A. Fattah Shaikh for his work on the PCI Industry Handbook Committee; Dr. Rolf Eligehausen and Dr. Werner Fuchs for developing the CCD method; and Dr. John E. Breen for leading the effort to incorporate design of anchorage to concrete into the ACI 318 building code. Without the tireless effort of Dr. Breen, the provisions would not have been at this advanced stage of development. Sincere gratitude is due to Dr. Breen.

This publication would not have been possible without the permission of the American Concrete Institute to reproduce copyrighted material in Appendices A and B. Appendices A and B of this document are duplicated from the proposed code and commentary, respectively, of Appendix D of ACI 318. Proposed Appendix D continues to be processed under the ACI consensus process. Appendices A and B are not an official ACI document, but rather part of a document being processed as an ACI consensus standard. The document, as well as the final consensus standard will be the copyright of ACI.

It has been a pleasure working with Dr. Ronald A Cook on the preparation of this document. His enthusiasm, thoroughness and professionalism are heartily acknowledged. We are glad he could fulfill “a career goal of providing designers with some‘building code based’ guidance on the design of anchorage to concrete.”

All figures included in Appendix B of this publication were provided courtesy of Dr. Breen and his team at the University of Texas at Austin. To all, a big “Thank you.”

Cami L. Cacciatore was responsible for the desktop publishing, including layout and formatting, of this manuscript. Her help in the production of this publication is gratefully acknowledged.

Basile G. Rabbat

About the Author

Dr. Ronald A. Cook has been involved with anchorage to concrete for over 25 years. In 1974, he worked for Dr. E. G. Burdette at the University of Tennessee on an anchorage research project sponsored by the Tennessee Valley Authority (TVA), and directed by Mr. Robert Cannon. The results of this project helped provide the basis for the 45-deg cone method presented in the 1976 edition of ACI 349. After obtaining his BSCE from the University of Tennessee in 1975, Dr. Cook was employed by the TVA where a large portion of his work involved the design of anchorage to concrete. During his employment with the TVA, Dr. Cook attended the University of Tennessee during evening hours and earned his MS in 1981. After leaving TVA, Dr. Cook founded a design firm in Knoxville, Tennessee and practiced as a consulting engineer until 1986 when he returned to school for his Ph.D. at the University of Texas at Austin. Dr. Cook's doctoral research dealt with developing a design standard for anchorage to concrete for the Texas DOT and developing a design model for multiple-anchor connections under the direction of Dr. Richard E. Klingner. Upon completing his Ph.D., Dr. Cook joined the faculty at the University of Florida where he has conducted several research projects related to anchorage to concrete, primarily in the area of adhesive and grouted anchors. Dr. Cook has been a member of ACI 355 (Anchorage to Concrete) and ACI 349 (Concrete Nuclear Structures) since 1990, a member of *fib* Task Group III/5 (Fastening to Reinforced Concrete and Masonry Structures) since 1992, and a member of ACI 318 Subcommittee B (Reinforcement and Development) since 1997.

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Strength Design of Anchorage to Concrete

INTRODUCTION

As of the late 1990's, the American Concrete Institute Building Code Requirements for Structural Concrete (ACI 318-99) and the American Institute of Steel Construction LRFD and ASD Specifications were silent regarding the design of anchorage (fastening) to concrete. ACI 349 Appendix B[1] and the PCI Design Handbook[2] have been the primary sources of design information for fastening attachments to concrete using cast-in-place anchors (fasteners). The design of connections to concrete using post-installed anchors has typically been based on information provided by individual anchor manufacturers.

During the past several years, ACI Committee 318 (ACI 318), titled Standard Building Code, has taken the lead in developing building code provisions for the design of anchorages to concrete using both cast-in-place and post-installed mechanical anchors. Committee 318 received support from ACI Committee 355 (ACI 355), Anchorage to Concrete, and ACI Committee 349, Concrete Nuclear Structures. Concurrent with the ACI 318 effort to develop design provisions, ACI 355 has been involved with developing a provisional test method for evaluating the performance of post-installed mechanical anchors in concrete. During the code cycle leading to ACI 318-99, a proposed Appendix D to ACI 318 dealing with the design of anchorages to concrete using both cast-in-place and post-installed mechanical anchors was approved by ACI 318. Final adoption of the proposed appendix was awaiting ACI 355 approval of a provisional test method for evaluating the performance of post-installed mechanical fasteners in concrete, and further reviews and public scrutiny, under the ACI consensus process.

Since ACI 355 was not able to complete the provisional test method for post-installed mechanical anchors on time to meet the publication deadlines for the ACI 318-99 code, an attempt was made to process an ACI 318 Appendix D reduced in scope to only cast-in-place anchors (i.e., without post-installed mechanical anchors). However, there was not sufficient time to meet the deadlines established by the International Code Council for submittal of the published ACI 318-99 standard to be referenced in the International Building Code (IBC 2000.) As a result, the anchorage to concrete provisions originally intended for ACI 318-99 Appendix D (excluding provisions for post-installed mechanical anchors) were submitted and approved for incorporation into IBC 2000 with some minor editorial changes and a technical change related to load combinations to be used when earthquake loads are present. These design provisions are given in Section 1913 of IBC 2000. The editorial differences relate to the order of presentation of the Scope, Definitions, and Notations sections, the use of the words "anchor" and "anchorage" in place of "fastener" and "fastening," and references to Section 1616 of IBC 2000 for seismic design categories.

It is anticipated that Section 1913 of IBC 2000 provisions will eventually be replaced by referencing ACI 318-02 Appendix D that will cover both cast-in-place and post-installed mechanical fasteners. It should be noted that research is in progress so that potentially ACI 318-05 will also cover post-installed adhesive and grouted fasteners.

HISTORICAL BACKGROUND OF DESIGN METHODS

The 45-deg cone method used in ACI 349 Appendix B[1] and the PCI Design Handbook[2] (referred to below as ACI 349/PCI) was developed in the mid 1970's. The model was based on tests conducted on cast-in-place anchors, at the University of Tennessee for the Tennessee Valley Authority. In the 1980's, comprehensive tests of different types of anchors with various embedment lengths, edge distances, and group effects were performed at the University of Stuttgart on both uncracked and cracked concrete. The Stuttgart test results led to the development of the Kappa (*K*) method that was introduced to ACI 349 and ACI 355 in the late 1980's. In the early 1990's, the *K* method was improved, and made user-friendlier at the University of Texas at Austin. This effort resulted in the Concrete Capacity Design (CCD) method. During this same period, an international database was assembled. During the mid 1990's, the majority of the work of ACI Committees 349 and 355 was to evaluate both the CCD method and the 45-deg cone (ACI 349/PCI) method using the international database of test results. As a result of this evaluation, ACI Committees 318, 349, and 355 proceeded with implementation of the CCD method. The design provisions for proposed ACI 318 Appendix D are based on the CCD method. Differences between the CCD method and the 45-deg cone (ACI 349/PCI) method are discussed below.

GENERAL CONSIDERATIONS

The design of anchorages to concrete must address both strength of the steel anchors and that associated with the embedded portion of the anchors. The lesser of these two strengths will control the design.

The strength of the steel portion of the anchor depends on the steel properties and size of the anchor. The strength of the embedded portion of the anchorage depends on its embedment length, strength of the concrete, proximity to other anchors, distance to free edges, and size of head (or hook) at the embedded end of the fastener.

The primary difference between the proposed ACI 318 Appendix D (IBC 2000 Section 1913) provisions and those given in ACI 349/PCI is the calculation of the embedment capacity for concrete breakout (i.e., a concrete cone failure). In the ACI 349/PCI method, the calculation of breakout capacity is based on a 45-deg concrete cone failure model that results in an equation based on the embedment length squared (h_{ef}^2). The proposed ACI 318 Appendix D (IBC 2000 Section 1913) provisions account for fracture mechanics and result in an equation for concrete breakout that is based on the embedment length to the 1.5 power ($h_{ef}^{1.5}$). Although the 45-deg concrete cone failure model gives conservative results for anchors with $h_{ef} \leq 6$ in., the proposed ACI 318 Appendix D (IBC 2000 Section 1913) provisions have been shown to give a better prediction of embedment strength for both single anchors and for anchors influenced by edge and group effects.

In addition to better prediction of concrete breakout strength, the proposed ACI 318 Appendix D (IBC 2000 Section 1913) provisions simplify the calculation of the effects of anchor groups and edges by using a rectangular area bounded by $1.5h_{ef}$ from each fastener and free edges rather than the overlapping circularcone areas used by ACI 349 Appendix B.

Appendix A of this publication is a reproduction of the proposed ACI 318 Appendix D code provisions for cast-in-place anchors, with prefix "A" used in place of "D" before each section and equation number. Appendix B

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of this publication is a reproduction of the commentary to proposed ACI 318 Appendix D for anchorages using cast-in-place anchors, with prefix "B" used in place of "RD".

With regard to the determination of anchor strength, Appendix A is technically identical to Section 1913 of IBC 2000 but does contain some editorial changes. With regard to load combinations, there is a technical difference between Appendix A Section A.3.2 and IBC 2000 Section 1913.3.2 concerning load combinations to be used when earthquake loads are present. Appendix A design provisions are intended for use with the ACI 318 strength design (factored loads) method of design. Since Allowable Stress Design (ASD) is not addressed in Appendix A, the IBC Section 1912 should be used for ASD.

DISCUSSION OF DESIGN PROVISIONS

The following provides a section-by-section discussion of the highlights of design provisions of Appendix A and of this publication. The section, equation, and figure numbers in the following discussion and examples refer to Appendices A and B. Where other parts of the ACI 318 Code or IBC 2000 are referenced, the section number is preceded by "ACI 318" or "IBC 2000".

Notation (A.0)

One primary editorial difference between the notation IBC Section 1913 and that of the proposed Appendix D of ACI 318 (Appendix A of this publication) is the substitution of the words "anchor" and "anchorage" for "fastener" and "fastening", respectively. Although most designers are familiar with the terms "anchor" and "anchorage" in the context of attaching steel to concrete, ACI 318-95 (and earlier editions of the code) contained a definition of "anchorage" that refers to end attachments in post-tensioned concrete applications. As a result, the proposed version of ACI 318 Appendix D uses the words "fastener" and "fastening" while IBC Section 1913 uses the words "anchor" and "anchorage". The remainder of this publication uses the proposed ACI 318 Appendix D terms.

Definitions (A.1)

Most of the definitions presented in Section A.1 are self-explanatory. The definitions of *brittle* and *ductile steel elements* are included to alert the designer that all types of steel are not necessarily ductile. As shown in Table 1, about one half of typical anchor materials satisfy the *ductile steel element* requirements. When using other types of anchor materials than those given in Table 1, the designer should refer to the appropriate material specification to be sure the material falls within the *ductile steel element* definition. Some high strength materials may not meet this requirement and must be considered as *brittle steel elements.*

Table 1 Summary of Mechanical Properties for Fastening Materials

Material Specification1	Grade or Type	Diameter	Tensile Strength, min (ksi)	Yield Strength, min		Elongation, min		Reduction of area, min, (%)
				ksi	method	%	length	
AWS D1.1[2]	B	1/2 -1	60	50	0.2%	20	2”	50
ASTM A 307[3]	A C	≤ 4 ≤ 4	60 58-80	--- 36	--- ---	18 23	2” 2”	--- ---
ASTM A 354[4]	BC BD	≤ 4 ≤ 4	125 1450	109 130	0.2% 0.2%	16 14	2” 2”	50 40
ASTM A 449[5]	1	≤ 1 1 – 1-1/2 > 1-1/2	120 108 90	92 81 58	0.2% 0.2% 0.2%	14 14 14	4D 4D 4D	35 35 35
ASTM A 687[6]		5/8 - 3	---	105	---	15	2”	45
ASTM F 1554[7]	36 55 105	≤ 2 ≤ 2 ≤ 2	58-80 75-95 125-150	36 55 105	0.2% 0.2% 0.2%	23 21 15	2” 2” 2”	40 30 45

Notes:

1. The materials listed are commonly used for concrete fasteners (anchors). Although other materials may be used (e.g., ASTM A 193 for high temperature applications, ASTM A 320 for low temperature applications), those listed are preferred for normal use. Structural steel bolting materials such as ASTM A 325 and ASTM A 490 are not typically available in the lengths needed for concrete fastening applications.
2. *Structural Welding Code - Steel* - This specification covers welded headed studs or welded hooked studs (unthreaded). None of the other listed specifications cover welded studs.
3. *Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength* - This material is commonly used for concrete fastening applications. Grade C is equivalent to ASTM A 36 steel.
4. *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners* - The strength of Grade BD is equivalent to ASTM A 490.
5. *Standard Specification for Quenched and Tempered Steel Bolts and Studs* - This specification is referenced by ASTM A 325 for “equivalent” anchor bolts.
6. *Standard Specification for High-Strength Nonheaded Steel Bolts and Studs* - This specification covers high-strength fasteners for anchorage applications with enhanced Charpy V-notch properties. The material does not have a minimum specified tensile strength (maximum is given as 150 ksi).
7. *Standard Specification for Anchor Bolts* - This specification covers straight and bent, headed and headless, anchor bolts in three strength grades. Anchors are available in diameters ≤ 4” but reduction in area requirements vary for anchors > 2 in.

The *5 percent fractile* is used to determine the nominal strength of the anchor. It represents a value such that if 100 anchors are tested there is a 90% confidence that 95 of the anchors will exhibit strengths higher than the 5 percent fractile value. The *5 percent fractile* is analogous to the use of f'_c for concrete strength and f_y for steel strength in the nominal strength calculations in other parts of the ACI 318 code. For example, ACI 318 Section 5.3 requires that the average compressive strength of the concrete f'_{cr} be statistically greater than the specified value of f'_c used in design calculations. For steel, f_y represents the specified yield strength of the material. Since ASTM specifications give the minimum specified yield strength, the value of f_y used in design is in effect a zero percent fractile (i.e., the designer is ensured that the actual steel used will have a yield value higher than the minimum specified value). All embedment strength calculations in Appendix A are based on a nominal strength calculated using *5 percent fractile* values (e.g., the k values used in calculating basic concrete breakout strength are based on the *5 percent fractile*).

Scope (A.2)

The provisions in the scope section only apply to cast-in-place fasteners (such as those illustrated in Fig. B.1 of Appendix B.) They include headed studs, headed bolts, and hooked rods (J and L bolts.) The design provisions apply to anchorages loaded with relatively static loads (i.e, fatigue and impact loads are not covered). All types of post-installed fasteners (i.e., mechanical, adhesive, grouted, pneumatically actuated nails or bolts) are currently excluded from the scope of Appendix A.

Section A.2.3 is primarily concerned with ensuring that the bolt or stud head size, or the rod hook size of cast-in-place fastener is sufficiently large to preclude a pullout failure. A pullout failure is where the fastener head or hook pulls out of the concrete prior to a full concrete cone breakout. Generally, pullout failures will not control for any standard bolt, headed stud, or hooked fastener. Anchors with pullout strength calculated in accordance with Section A.5.3 that exceed the strength of the lowest calculated failure load associated with other failure modes satisfy this requirement.

General requirements (A.3)

The analysis methods prescribed in Section A.3 to determine loads on individual fasteners in multiple fastener applications depend on the type of loading, rigidity of the attachment base plate, and the embedment of the fasteners.

For multiple-fastener connections loaded concentrically in pure tension, the applied tensile load may be assumed to be evenly distributed among the fasteners if the base plate has been designed so as not to yield. Prevention of yielding in the base plate will ensure that prying action does not develop in the connection.

For multiple-fastener connections loaded with an eccentric tension load or moment, distribution of loads to individual fasteners should be determined by elastic analysis unless calculations indicate that sufficient ductility exists in the embedment of the fasteners to permit a redistribution of load among individual fasteners. If sufficient ductility is provided, a plastic design approach may be used. The plastic design approach requires ductile steel fasteners sufficiently embedded so that embedment failure will not occur prior to a ductile steel failure. The plastic design approach assumes that the tension load (either from eccentric tension or moment) is equally distributed among the tension fasteners. For connections subjected to moment, the plastic design

approach is analogous to multiple layers of flexural reinforcement in a reinforced concrete beam. If the multiple layers of steel are adequately embedded and are a sufficient distance from the neutral axis of the member, they may be considered to have reached yield.

For both the elastic and plastic analysis methods of multiple-fastener connections subjected to moment, the exact location of the compressive resultant cannot be accurately determined by traditional concrete beam methods. This is true for both the elastic linear stress-strain method (i.e., the transformed area method) and the ACI 318 stress block method since plane sections do not remain plane. For design purposes, the compression resultant from applied moment may be assumed to be located one base plate thickness away from the compression element of the attached member unless base plate stiffeners are provided. If base plate stiffeners are provided, the compressive resultant may be assumed to be located at the leading edge of the base plate.

The load combinations of ACI 318 Section 9.2 should be used unless earthquake loads are included in which case the load combinations of IBC 2000 Section 1605.2 should be used. Note that the requirement to use IBC 2000 Section 1605.2 for combinations with earthquake loads is a specific requirement of IBC 2000 Section 1913.3.2 and is not given in Appendix A of this publication (i.e., the proposed ACI 318 version).

For fastener design in zones of moderate or high seismic risk (IBC 2000 Seismic Design Category C, D, E or F), all values for ϕN_n and ϕV_n must be multiplied by an additional reduction factor of 0.75. Further, the strength of the connection must be controlled by the strength of ductile steel elements and not the embedment strength or the strength of brittle steel elements. Commentary Section B.3.3 provides a detailed discussion of these requirements.

General requirements for strength of structural fasteners (A.4)

The general requirements section provides a general discussion of the failure modes that must be considered in the design of fastenings to concrete. The section also provides capacity reduction factors, ϕ, for each type of failure mode. The failure modes that must be considered include those related to the steel strength and those related to the strength of the embedment.

Failure modes related to steel strength are simply tensile failure [Figure B.4.1A(a)] and shear failure [Figure B.4.1B(a)] of the fastener steel. Fastener steel strength is relatively easy to compute but typically does not control the design of the connection unless there is a specific requirement that the steel strength of a ductile steel element controls the design.

Embedment failure modes that must be considered are illustrated in Appendix Figs. B.4.1A and B.4.1B. They include:

- *concrete breakout* - a concrete cone failure emanating from the head of tension fasteners [Figure B.4.1A(c)] or from the entry point of shear fasteners located near an edge [Figure B.4.1B(c)]
- *pullout* - a straight pullout of the fastener such as might occur for a fastener with a small head [Figure B.4.1A(b)]
- *side-face blowout* - a spalling at the embedded head of fasteners located near a free edge [Figure B.4.1A(d)]

- *concrete pryout* - a shear failure mode that can occur with a short fastener popping out a wedge of concrete on the back side of the fastener [Figure B.4.1B(b)]
- *splitting* - a tensile failure mode related to fasteners placed in relatively thin concrete, [Figure B.4.1A(e)]

The use of any design model that results in predictions of strength that are in substantial agreement with test results is also permitted by the general requirements section. If the designer feels that the provisions of ACI 349 Appendix B[1], the PCI Design Handbook[2], or any other method satisfy this requirement he or she is permitted to use them. If not, the design provisions of the remaining sections of Appendix A should be used provided the fastener diameter does not exceed 2 in. and the embedment length does not exceed 25 in.

Design requirements for tensile loading (A.5)

Methods to determine the nominal tensile strength as controlled by steel strength and embedment strength are presented in the section on tensile loading. The nominal tensile strength of the steel is based on either the yield strength, *Eq. (A-3) of Appendix A,* or the ultimate strength, *Eq. (A-4),* of the steel depending on the properties of the steel material. The nominal tensile strength of the embedment is based on (1) concrete breakout strength, *Eq. (A-5a)* for single fasteners or *Eq. (A-5b)* for groups of fasteners, (2) pullout strength, *Eq. (A-10),* or (3) side-face blowout strength, *Eq. (A-12)* for single fasteners or *Eq. (A-13)* for groups. When combined with the appropriate capacity reduction factors from Section A.4.4, the smaller of these strengths will control the design tensile capacity of the anchorage.

Steel strength of fastener in tension (A.5.1)

Table 1 provides values for f_y and f_{ut} for typical fastener materials. Note that only ASTM A 307 Type C and ASTM A 687 have a well defined yield point, all other fastening materials have a minimum specified yield strength based on 0.2% offset. This method of determining yield strength does not indicate a well-defined yield point. For the fastener materials listed in Table 1 (other than ASTM A 307 Type C and ASTM A 687), only *Eq. (A-4)* is applicable.

For standard fasteners (i.e., threaded fasteners, headed studs and hooked bars), the effective cross-sectional area of the fastener (A_{se}) is the net tensile stress area for threaded fasteners and the gross area for headed studs that are welded to a base plate. These areas are provided in Table 2. For fasteners of unusual geometry, the nominal steel strength may be taken as the lower 5% fractile of test results.

Table 2 Summary of dimensional properties of fasteners

Fastener Diameter (in.)	Gross Area of Fastener (in.²)	Effective Area of Threaded Fastener (in.²)	Bearing Area of Heads, Nuts, and Washers (A_b) (in.²)				
			Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	2.405	1.900	-	-	-	4.144	6.541
2.000	3.142	2.500	-	-	-	5.316	7.903

Concrete breakout strength of fastener in tension (A.5.2)

Figure B.4.1A(c) shows a typical concrete breakout failure (i.e., concrete cone failure) for a single headed cast-in-place anchor loaded in tension. *Eq. (A-5a)* gives the concrete breakout strength for a single anchor while Eq. (A-5b) gives the concrete breakout strength for a group of anchors in tension.

The individual terms in *Eq. (A-5a)* and *Eq. (A-5b)* are discussed below:

N_b: The basic concrete breakout strength for a single anchor located away from edges and other anchors (N_b) is given by *Eq. (A-7a)* or *Eq. (A-7b)*. As previously noted, the primary difference between these equations and those given in ACI 349 Appendix B[1] and the PCI Design Handbook[2] (i.e., the 45-deg concrete cone failure) is the use of $h_{ef}^{1.5}$ in Eq. (A-7a) (or alternatively $h_{ef}^{1.67}$ for anchors with $h_{ef} \geq 11$ in. in *Eq. (A-7b))* rather than h_{ef}^{2}. The use of $h_{ef}^{1.5}$ accounts for fracture mechanics principles and can be thought of as follows:

$$N_b = \frac{k\sqrt{f_c'}\,h_{ef}^2}{h_{ef}^{0.5}}\left[\frac{\textit{general } 45^\circ \textit{ concrete cone equation}}{\textit{modification factor for fracture mechanics}}\right]$$

Resulting in:

$$N_b = k\sqrt{f_c'}\,h_{ef}^{1.5} \qquad \textit{Eq. (A-7a)}$$

The fracture mechanics approach accounts for the high tensile stresses that exist at the embedded head of the anchor while other approaches (ACI 349/PCI) assume a uniform distribution of stresses over the assumed failure surface.

The numeric constant $\boldsymbol{k}$ of 24 in *Eq. (A-7a)* (or $\boldsymbol{k}$ of 16 in Eq. A-7b if $\boldsymbol{h_{ef}} \geq 11$ in.) is based on the 5% fractile of test results on headed cast-in-place anchors in cracked concrete. These $\boldsymbol{k}$ values must be used unless special testing has shown that higher values of k are applicable. Note that the crack width used in tests to establish these $\boldsymbol{k}$ values was 0.012 in. If larger crack widths are anticipated confining reinforcement to control crack width to about 0.012 in. should be provided or special testing in larger cracks should be performed.

$\frac{A_N}{A_{No}}$: This factor accounts for adjacent anchors and/or free edges. For a single anchor located away from free edges, the $\boldsymbol{A_{No}}$ term is the projected area of a 35-deg failure plane (measured from the horizontal) at the surface of the concrete and defined by a square with the sides **1.5** $\boldsymbol{h_{ef}}$ from the centerline of the anchor [Figure B.5.1(a)]. The $\boldsymbol{A_N}$ term is a rectilinear projected area of the 35-deg failure plane at the surface of the concrete with sides **1.5** $\boldsymbol{h_{ef}}$ from the centerline of the anchor(s) as limited by adjacent anchors and/or free edges. The definition of $\boldsymbol{A_N}$ is shown by Figure B.5.1(b).

Ψ_1: This factor is applicable when multiple rows of tension anchors are present and the elastic design approach is used. In this case, the individual rows of tension anchors are assumed to carry different levels of load with the centerline of action of the applied tension load at an eccentricity (e_n') from the centroid of the tension anchors. If the plastic design approach is used, all tension anchors are assumed to carry the same load and the eccentricity factor is taken as 1.0.

Ψ_2: This factor accounts for the non-uniform distribution of stresses when an anchor is located near a free edge of the concrete that are not accounted for by the $\frac{A_N}{A_{No}}$ term.

Ψ_3: This factor is taken as 1.0 if cracks in the concrete are likely to occur at the location of the anchor(s). If calculations indicate that concrete cracking is not likely to occur (e.g., $\boldsymbol{f_t} < \boldsymbol{f_r}$), then ψ_3 may be taken as 1.25.

Pullout strength of fastener in tension (A.5.3)

A schematic of the pullout failure mode is shown in Figure B.4.1A(b). The pullout strength of the fastener is related to the bearing area ($\boldsymbol{A_b}$) at the embedded end of headed fasteners and the properties of embedded hooks ($\boldsymbol{e_h}$ and $\boldsymbol{d_o}$) for J-bolts and L-bolts. Obviously, if a fastener has no head or hook it will simply pull out of the concrete and not be able to achieve the concrete breakout strength associated with a full concrete cone failure (Section A.5.2). With an adequate head or hook size, pullout will not occur and the concrete breakout strength can be achieved. *Eq. (A-10)* provides the general requirement for pullout while *Eq. (A-11a)* and *Eq. (A-11b)*

provide the specific requirements for headed and hooked fasteners, respectively.

Note that the bearing area of the embedded head (A_b) is the gross area of the head less the gross area of the fastener (i.e., not the area of the embedded head). Table 2 provides values for A_b for standard bolt heads, nuts, and washers.

Concrete side-face blowout strength of headed fastener in tension (A.5.4)

The side-face blowout strength is associated with the lateral pressure that develops around the embedded end of headed fasteners under load. When the minimum edge distance for a single headed fastener is less than $0.4h_{ef}$, side-face blowout must be considered using *Eq. (A-12)*. If an orthogonal free edge (i.e., a fastener in a corner) is located less than three times the minimum edge distance (i.e., the distance from the fastener to the nearest edge) then an additional reduction factor of $((1+ \mathbf{c_{orthognal}}/\mathbf{c_{min}})/4)$ must be applied to *Eq. (A-12)*.

For multiple fastener groups, the side-face blowout strength is that given by *Eq. (A-12)* provided the spacing between individual fasteners parallel to a free edge is greater than or equal to six times the distance to the free edge. If the spacing of the fasteners in the group is less than six times the distance to the free edge, *Eq. (A-13)* must be used.

Design requirements for shear loading (A.6)

Methods to determine the nominal shear strength as controlled by steel strength and embedment strength are specified in Section A.6 . The nominal shear strength of the steel is based on either the yield strength *(Eq. A-14)* or the ultimate strength *(Eq. A-15)* of the steel depending on the properties of the steel material. The nominal shear strength of the embedment is based on concrete breakout strength *(Eq. (A-16a)* for single fasteners or *Eq. (A-16b)* for groups of fasteners) or pryout strength *(Eq. (A-21))*. When combined with the appropriate capacity reduction factors from Section A.4.4, the smaller of these strengths will control the design shear capacity of the anchorage.

Steel strength of fastener in shear (A.6.1)

Table 1 provides values for f_y and f_{ut} for typical fastener materials. Note that only ASTM A 307 Type C and ASTM A 687 have a well defined yield point, all other fastening materials have a minimum specified yield strength based on 0.2% offset. This method of determining yield strength does not indicate a well defined yield point. For the fastener materials listed in Table 1 (other than ASTM A 307 Type C and ASTM A 687), only *Eq. (A-15)* is applicable.

For most applications the effective cross-sectional area of the fastener for shear (A_{se}) should be taken as the net tensile stress area for threaded fasteners and the gross area for headed studs. These areas are given in Table 2. If the threads of headed fasteners are located well above the shear plane (at least two diameters) the gross area of the fastener may be used for shear. For fasteners of unusual geometry, the nominal steel strength may be taken as the lower 5% fractile of test results.

When built-up grout pads are present, the shear strength values given by *Eq. (A-14)* and *Eq. (A-15)* must be

reduced by 20% to account for the flexural stresses developed in the anchor if the grout pad fractures upon application of the shear load.

Concrete breakout strength of fastener in shear (A.6.2)

Figure B.4.1B(c) shows typical concrete breakout failures for anchors loaded in shear directed toward a free edge. *Eq. (A-16a)* gives the concrete breakout strength for any single anchor while *Eq. (A-16b)* gives the concrete breakout strength for groups of anchors in shear.

The individual terms in *Eq. (A-16a)* and *Eq. (A-16b)* are discussed below:

V_b: The basic concrete breakout strength for a single anchor loaded in shear, directed toward a free edge (V_b) without any other adjacent free edges or limited concrete thickness is given by Eq. (A-18a) for typical bolted connections and *Eq. (A-18b)* for connections with welded studs or other anchors welded to the attached base plate. The primary difference between these equations and those given in ACI 349 Appendix B[1] and the PCI Design Handbook[2] (i.e., the 45-deg concrete cone failure) is the use of $c_1^{1.5}$ rather than c_1^{2}. The use of $c_1^{1.5}$ accounts for fracture mechanics principles in the same way that $h_{ef}^{1.5}$ does for tension anchors. The fracture mechanics approach accounts for the high tensile stresses that exist in the concrete at the point where the anchor first enters the concrete.

ℓ, d_o: The terms involving ℓ and d_o in *Eq. (A-18a)* and *Eq. (A-18b)* relate to the shear stiffness of the anchor. A stiff anchor is able to distribute the applied shear load further into the concrete than a flexible anchor.

$\frac{A_V}{A_{Vo}}$: This factor accounts for adjacent anchors, concrete thickness, and free edges. For a single anchor in thick concrete member with shear directed toward a free edge, the A_{Vo} term is the projected area on the side of the free edge of a 35-deg failure plane radiating from the point where the anchor first enters the concrete and directed toward the free edge (see Figure B.6.2(a)). The A_V term is a rectilinear projected area of the 35-deg failure plane on the side of the free edge with sides $1.5c_1$ from the point where the anchor first enters the concrete as limited by adjacent anchors, concrete thickness and free edges. The definition of A_V is shown in Figure B.6.2(b).

Ψ_5: This factor applies when the applied shear load does not act through the centroid of the anchors loaded in shear [see Figure B.6.2(e)].

Ψ_6: This factor accounts for the non-uniform distribution of stresses when an anchor is located in a corner that is not accounted for by the $\frac{A_V}{A_{Vo}}$ term [see Figure B.6.2(f)].

Ψ_7: This factor is taken as 1.0 if cracks in the concrete are likely to occur at the location of the anchor(s) and no supplemental reinforcement has been provided. If calculations indicate that concrete cracking is not likely to occur (e.g., $f_t < f_r$ at service loads), then Ψ_7 may be taken as 1.4. Values of $\Psi_7 > 1.0$ may be used if cracking at service loads is likely, provided No. 4 bar edge reinforcement is provided (see A.6.2.7).

Concrete pryout strength of fastener in shear (A.6.3)

The concrete pryout strength of an anchor in shear may control when an anchor is both short and relatively stiff. Figure B.4.1B(b) shows this failure mode. As a mental exercise, this failure mode may be envisioned by thinking of a No. 8 bar embedded 2 in. in concrete with 3 ft of the bar sticking out. A small push at the top of the bar will cause the bar to "pryout" of the concrete.

Interaction of tensile and shear forces (A.7)

The interaction requirements for tension and shear are based on a tri-linear approximation to the following interaction equation (see Figure B.7):

$$\left[\frac{N_u}{\phi N_n}\right]^{\frac{5}{3}} + \left[\frac{V_u}{\phi V_n}\right]^{\frac{5}{3}} = 1$$

In the tri-linear simplification, Section A.7.1 permits the full value of ϕN_n if $V_u \leq 0.2\ \phi V_n$ and Section A.7.2 permits the full value of ϕV_n if $N_u \leq 0.2\ \phi N_n$. If neither of these conditions can be satisfied, the linear interaction of Eq. (A-22) must be used.

The most important aspect of the interaction provisions is that both $\phi \boldsymbol{N_n}$ and $\phi \boldsymbol{V_n}$ are the smaller of the fastening strength as controlled by the fastener steel or the embedment. Tests have shown that the interaction relationship is valid whether steel strength or embedment strength controls for $\phi \boldsymbol{N_n}$ or $\phi \boldsymbol{V_n}$.

Required edge distances, spacings, and thicknesses to preclude splitting failure (A.8)

Section A.8 addresses the required edge distances to account for post-installed mechanical fasteners with expansion devices at the embedded end of the fastener. Post-installed mechanical fasteners can exert large lateral pressures at the embedded expansion device during installation that can lead to a splitting failure. Cast-in-place fasteners are typically not highly torqued and the minimum cover requirements of ACI 318 Section 7.7 coupled with the side-face blow-out provisions of Section A.5.4 when headed fasteners are used will prevent splitting. For headed cast-in-place fasteners that will be torqued, a minimum edge distance of $\boldsymbol{6d_0}$ is required.

Section A.8.2 covers both cast-in-place and post-installed fasteners. For cast-in-place fasteners, the minimum cover requirements of ACI 318 Section 7.7 and side-face blowout strength of Section A.5.4 must be met (i.e., neglect Section A.8.2).

Installation of fasteners (A.9)

Cast-in-place fasteners should be installed in accordance with construction documents. For threaded fasteners, a metal or plywood template mounted above the surface of the concrete with nuts on each side of the template should be used to hold the anchors in a fixed position while the concrete is placed, consolidated, and hardens.

REFERENCES

1. ACI Committee 349, "Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-85), Appendix B - Steel Embedment," ACI Manual of Concrete Practice, Part 4, 1987.
2. PCI Design Handbook, 4th Edition, Precast/Prestressed Concrete Institute, Chicago, 1992, pp. 570.
3. ACI Committee 318, "Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99)", American Concrete Institute, Farmington Hills, Mich., 1999, 391 pp.

Appendix A - Design Provisions

A.0 – Notation

A_b = bearing area of the head of stud or anchor bolt, in^2.

A_{No} = projected concrete failure area of one fastener, for calculation of strength in tension, when not limited by edge distance or spacing, as defined in A.5.2.1, in^2. [See Fig. B.5.1(a)]

A_N = projected concrete failure area of a fastener or group of fasteners, for calculation of strength in tension, as defined in A.5.2.1, in^2. A_N shall not be taken greater than nA_{No}. [See Fig. B.5.1(b)]

A_{se} = effective cross-sectional area of fastener, in^2.

A_{Vo} = projected concrete failure area of one fastener, for calculation of strength in shear, when not limited by corner influences, spacing, or member thickness, as defined in A.6.2.1, in^2. [See Fig. B.6.2(a)]

A_V = projected concrete failure area of a fastener or group of fasteners, for calculation of strength in shear, as defined in A.6.2.1, in^2. A_V shall not be taken greater than nA_{Vo}. [See Fig. B.6.2(b)]

c = distance from center of a fastener shaft to the edge of concrete, in.

c_1 = distance from the center of a fastener shaft to the edge of concrete in one direction, in. Where shear force is applied to fastener, c_1 is in the direction of the shear force. [See Fig. B.6.2(a)]

c_2 = distance from center of a fastener shaft to the edge of concrete in the direction orthogonal to c_1, in.

c_{max} = the largest of the edge distances that are less than or equal to $1.5h_{ef}$, in. (used only for the case of 3 or 4 edges).

c_{min} = the smallest of the edge distances that are less than or equal to $1.5h_{ef}$, in.

d_o = shaft diameter of headed stud, headed anchor bolt, or hooked anchor, in. (See also A.8.2)

d_u = diameter of head of stud or anchor bolt or equivalent diameter of effective perimeter of an added plate or washer at the head of the fastener, in.

e_h = distance from the inner surface of the shaft of a J-bolt or L-bolt to the outer tip of the J- or L-bolt, in.

e_N' = eccentricity of normal force on a group of fasteners; the distance between the resultant tension load on a group of fasteners in tension and the centroid of the group of fasteners loaded in tension, in. [See Fig. B.5.2(b) and (c)]

e_V' = eccentricity of shear force on a group of fasteners; the distance between the point of shear force application and the centroid of the group of fasteners resisting shear in the direction of the applied shear, in.

f_c' = specified compressive strength of concrete, psi.

f_{ct} = specified tensile strength of concrete, psi.

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f_r = modulus of rupture of concrete, psi. (See 9.5.2.3*)

f_t = calculated tensile stress in a region of a member, psi.

f_y = specified yield strength of fastener steel, psi.

f_{ut} = specified tensile strength of fastener steel, psi.

h = thickness of member in which a fastener is anchored measured parallel to fastener axis, in.

h_{ef} = effective fastener embedment depth, in. (See Fig. B.1)

k = coefficient for basic concrete breakout strength in tension.

k_{cp} = coefficient for pryout strength.

ℓ = load bearing length of fastener for shear, not to exceed $8d_o$, in.
= h_{ef} for fasteners with a constant stiffness over the full length of the embedded section, such as headed studs

n = number of fasteners in a group.

N_b = basic concrete breakout strength in tension of a single fastener in cracked concrete, as defined in A.5.2.2, lb.

N_{cb} = nominal concrete breakout strength in tension of a single fastener, as defined in A.5.2.1, lb.

N_{cbg} = nominal concrete breakout strength in tension of a group of fasteners, as defined in A.5.2.1, lb.

N_n = nominal strength in tension, lb.

N_p = pullout strength in tension of a single fastener in cracked concrete, as defined in A.5.3.3 or A.5.3.4, lb.

N_{pn} = nominal pullout strength in tension of a single fastener, as defined in A.5.3.1, lb.

N_{sb} = side-face blowout strength of a single fastener, as defined in A.5.4.1, lb.

N_{sbg} = side-face blowout strength of a group of fasteners, as defined in A.5.4.2, lb.

N_s = nominal strength of a single fastener in tension as governed by the steel strength, as defined in A.5.1.2, lb.

N_u = factored tensile load, lb.

s = fastener center-to-center spacing, in.

s_o = spacing of the outer fasteners along the edge in a group, in.

t = thickness of washer or plate, in.

V_b = basic concrete breakout strength in shear of a single fastener in cracked concrete, as defined in A.6.2.2 or A.6.2.3, lb.

V_{cb} = nominal concrete breakout strength in shear of a single fastener, as defined in A.6.2.1, lb.

V_{cbg} = nominal concrete breakout strength in shear of a group of fasteners, as defined in A.6.2.1, lb.

**See ACI 318-99.*

V_{cp} = nominal concrete pryout strength, as defined in A.6.3, lb.

V_n = nominal shear strength, lb.

V_s = nominal strength in shear of a single fastener as governed by the steel strength, as defined in A.6.1.1, lb.

V_u = factored shear load, lb.

ϕ = strength reduction factor (see A.4.4 and A.4.5).

Ψ_1 = modification factor, for strength in tension, to account for fastener groups loaded eccentrically, as defined in A.5.2.4.

Ψ_2 = modification factor, for strength in tension, to account for edge distances smaller than $\mathbf{1.5}h_{ef}$, as defined in A.5.2.5.

Ψ_3 = modification factor, for strength in tension, to account for cracking, as defined in A.5.2.6 and A.5.2.7.

Ψ_4 = modification factor, for pullout strength, to account for cracking, as defined in A.5.3.1 and A.5.3.5.

Ψ_5 = modification factor, for strength in shear, to account for fastener groups loaded eccentrically, as defined in A.6.2.5.

Ψ_6 = modification factor, for strength in shear, to account for edge distances smaller than $\mathbf{1.5}c_1$ as defined in A.6.2.6.

Ψ_7 = modification factor, for strength in shear, to account for cracking, as defined in A.6.2.7.

A.1 — Definitions

Attachment – The structural assembly, external to the surface of the concrete, that transmits loads to the fastener.

Brittle Steel Element – An element with a tensile test elongation of less than 14 percent over a 2 in. gage length, reduction in area of less than 40 percent, or both.

Concrete Breakout Strength – The strength corresponding to a volume of concrete surrounding the fastener or group of fasteners separating from the member.

Concrete Pryout Strength – The strength corresponding to formation of a concrete spall behind a short, stiff fastener with an embedded base that is displaced in the direction opposite to the applied shear force.

Ductile Steel Element – An element with a tensile test elongation of at least 14 percent over a 2 in. gage length and reduction in area of at least 40 percent.

Edge Distance – The distance from the edge of the concrete surface to the center of the nearest fastener.

Effective Embedment Depth – The overall depth through which the fastener transfers force to the surrounding concrete. The effective embedment depth will normally be the depth of the failure surface in tension applications. For cast-in headed anchor bolts and headed studs, the effective embedment depth is measured from the bearing contact surface of the head. (See Fig. B.1)

Fastener – A metallic element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads including straight bolts, hooked bolts (J- or L-bolt), headed studs, expansion fasteners, undercut fasteners, or inserts.

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Fastener Group – A number of fasteners of approximately equal effective embedment depth with each fastener spaced at less than three times its embedment depth from one or more adjacent fasteners.

Fastener Pullout Strength – The strength corresponding to the fastening device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete.

5 Percent Fractile **–** A statistical term meaning 90 percent confidence that 95 percent of the actual strengths will exceed the nominal strength. Determination shall include the number of tests when evaluating data.

Hooked Bolt **–** A cast-in fastener anchored mainly by mechanical interlock from the 90-deg bend (L-bolt) or 180-deg bend (J-bolt) at its lower end.

Insert (specialty insert) – Predesigned and prefabricated cast-in fasteners specifically designed for attachment of bolted or slotted connections. Inserts are often used for handling, transportation, and erection, but are also used for fastening structural elements.

Post-Installed Fastener – A fastener installed in hardened concrete. Expansion fasteners and undercut fasteners are examples of post-installed fasteners.

Projected Area – The area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface.

Side-Face Blowout Strength – The strength of fasteners with deeper embedment but thinner side cover corresponding to concrete spalling on the side face around the embedded head while no major breakout occurs at the top concrete surface.

A.2 — Scope

A.2.1 – This Appendix provides design requirements for structural fasteners in concrete used to transmit structural loads from attachments into concrete members or from one connected concrete member to another by means of tension, shear or a combination of tension and shear. Safety levels specified are intended for in-service conditions, rather than for short-term handling and construction conditions.

A.2.2 – This Appendix applies only to cast-in fasteners such as headed studs, headed bolts or hooked bolts. Post-installed fasteners, such as expansion fasteners and undercut fasteners, specialty inserts, through bolts, bolts anchored to embedded large steel plates, adhesive or grouted fasteners, and direct fasteners such as powder or pneumatic actuated nails or bolts are not included. Reinforcement used as part of the embedment shall be designed in accordance with other parts of the code.

A.2.3 – Headed studs and headed bolts that have a geometry that has been demonstrated to result in a pullout strength in uncracked concrete equal or exceeding $\mathbf{1.4}\boldsymbol{N_p}$ (where $\boldsymbol{N_p}$ is given by Eq. (A-11a)) are included. Hooked bolts that have a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal or exceeding $\mathbf{1.4}\boldsymbol{N_p}$ (where $\boldsymbol{N_p}$ is given by Eq. (A-11b)) are included.

A.2.4 – Load applications that are predominantly high cycle fatigue or impact are not covered by this Appendix.

A.3 — General requirements

A.3.1 – Fasteners and fastener groups shall be designed for critical effects of factored loads as determined by elastic analysis. Plastic analysis approaches are permitted where nominal strength is controlled by ductile steel elements, provided that deformational compatibility is taken into account.

A.3.2 – Fasteners shall be designed for all load combinations outlined in 9.2*.

A.3.3 – When fastener design includes seismic loads the following additional requirements shall apply.

A.3.3.1 – In regions of intermediate or high seismic risk or for structures assigned to intermediate or high seismic performance or design categories, the design strength of fasteners shall be taken as $\mathbf{0.75\phi N_n}$ and $\mathbf{0.75\ \phi V_n}$, where ϕ is given in A.4.4 or A.4.5 and $\boldsymbol{N_n}$ and $\boldsymbol{V_n}$ are determined in accordance with A.4.1.

A.3.3.2 – In regions of intermediate or high seismic risk or for structures assigned to intermediate or high seismic performance or design categories, fasteners shall be designed to be governed by tensile or shear strength of a ductile steel element, unless A.3.3.3 is satisfied.

A.3.3.3 – In lieu of A.3.3.2, the attachment that the fastener is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level no greater than 75 percent of the minimum fastener design strength.

A.3.4 – All provisions for fastener axial tension and shear strength apply to normal weight concrete. When lightweight aggregate concrete is used, provisions for $\boldsymbol{N_n}$ and $\boldsymbol{V_n}$ shall be modified by multiplying all values of $\sqrt{f'_c}$ affecting $\boldsymbol{N_n}$ and $\boldsymbol{V_n}$ by 0.75 for "all-lightweight" concrete and 0.85 for "sand-lightweight" concrete. Linear interpolation shall be permitted when partial sand replacement is used.

A.3.5 – The values of $\boldsymbol{f'_c}$ used for calculations in this Appendix shall not exceed 10,000 psi.

A.4 — General requirements for strength of structural fasteners

A.4.1 – Strength design of structural fasteners shall be based on the computation or test evaluation of the following:

a) steel strength of fastener in tension (A.5.1)
b) steel strength of fastener in shear (A.6.1)
c) concrete breakout strength of fastener in tension (A.4.2 and A.5.2)
d) concrete breakout strength of fastener in shear (A.4.2 and A.6.2)
e) pullout strength of fastener in tension (A.4.2 and A.5.3)
f) concrete side-face blowout strength of fastener in tension (A.4.2 and A.5.4)
g) concrete pryout strength of fastener in shear (A.4.2 and A.6.3)
h) required edge distances, spacings and thicknesses to preclude splitting failure (A.4.2 and A.8).

A.4.1.1 – For the design of fasteners, except as required in A.3.3:

$$\phi N_n \geq N_u \tag{A-1}$$

$$\phi V_n \geq V_u \tag{A-2}$$

A.4.1.2 – When both $\boldsymbol{N_u}$ and $\boldsymbol{V_u}$ are present, interaction effects shall be considered in accordance with A.4.3.

A.4.1.3 – In Eq. (A-1) and (A-2), $\boldsymbol{\phi N_n}$ and $\boldsymbol{\phi V_n}$ are the lowest design strengths determined from all appropriate failure modes. $\boldsymbol{\phi N_n}$ is the lowest design strength in tension of a fastener or group of fasteners as determined from consideration of $\boldsymbol{\phi N_s}$, $\boldsymbol{\phi N_{pn}}$, either $\boldsymbol{\phi N_{sb}}$ or $\boldsymbol{\phi N_{sbg}}$, and either $\boldsymbol{\phi N_{cb}}$ or $\boldsymbol{\phi N_{cbg}}$. $\boldsymbol{\phi V_n}$ is the lowest design strength in shear of a fastener or a group of fasteners as determined from consideration of $\boldsymbol{\phi V_s}$, either $\boldsymbol{\phi V_{cb}}$ or $\boldsymbol{\phi V_{cbg}}$, and $\boldsymbol{\phi V_{cp}}$.

**See ACI 318-99.*

A.4.2 – The nominal strength for any fastener or group of fasteners shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests and that account for size effects. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent fractile of the basic individual fastener strength, with modifications made for the number of fasteners, the effects of close spacing of fasteners, proximity to edges, depth of the concrete member, eccentric loadings of fastener groups, and presence or absence of cracking. Limits on edge distances and fastener spacing in the design models shall be consistent with the tests that verified the model.

A.4.2.1 – The effect of supplementary reinforcement provided to confine or restrain the concrete breakout, or both, shall be permitted to be included in the design models of A.4.2.

A.4.2.2 – For fasteners with diameters not exceeding 2 in., and tensile embedments not exceeding 25 in. in depth, the concrete breakout strength requirements of A.4.2 shall be considered satisfied by the design procedure of A.5.2 and A.6.2.

A.4.3 – Resistance to combined tensile and shear loads shall be considered in design using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by A.7.

A.4.4 – Strength reduction factor ϕ for fastening to concrete shall be as follows when the load combinations of 9.2* are used:

a) Fastener governed by tensile or shear strength of a ductile steel element **0.90**

b) Fastener governed by tensile or shear strength of a brittle steel element **0.75**

c) Fastener governed by concrete breakout, blowout, pullout or pryout strength

		Condition A	Condition B
i)	Shear Loads	**0.85**	**0.75**
ii)	Tension Loads	**0.85**	**0.75**

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided or where pullout or pryout strength governs.

A.4.5 – Strength reduction factor ϕ for fastening to concrete shall be as follows when the load combinations referenced in Appendix C* are used:

a) Fastener governed by tensile or shear strength of a ductile steel element **0.80**

b) Fastener governed by tensile or shear strength of a brittle steel element **0.70**

c) Fastener governed by concrete breakout, blowout, pullout or pryout strength

		Condition A	Condition B
i)	Shear Loads	**0.75**	**0.70**
ii)	Tension Loads	**0.75**	**0.70**

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided or where pullout or pryout strength governs.

**See ACI 318-99.*

A.5 — Design requirements for tensile loading

A.5.1 — Steel strength of fastener in tension

A.5.1.1 – The nominal strength of a fastener in tension as governed by the steel, N_s , shall be evaluated by calculations based on the properties of the fastener material and the physical dimensions of the fastener. Alternatively, it shall be permitted to use values based on the 5 percent fractile of test results to establish values of N_s.

A.5.1.2 – Unless determined by the 5 percent fractile of test results, nominal strength of a fastener or group of fasteners in tension shall not exceed:

(a) for fastener material with a well defined yield point:

$$N_s = nA_{se} f_y \quad \text{(A-3)}$$

(b) for fastener material without a well defined yield point where f_{ut} shall not be taken greater than 125,000 psi:

$$N_s = nA_{se} (0.8 f_{ut}) \quad \text{(A-4)}$$

A.5.2 — Concrete breakout strength of fastener in tension

A.5.2.1 – Unless determined in accordance with A.4.2, nominal concrete breakout strength of a fastener or group of fasteners in tension shall not exceed:

(a) for a fastener:

$$N_{cb} = \frac{A_N}{A_{No}} \psi_2 \psi_3 N_b \quad \text{(A-5a)}$$

(b) for a group of fasteners:

$$N_{cbg} = \frac{A_N}{A_{No}} \psi_1 \psi_2 \psi_3 N_b \quad \text{(A-5b)}$$

N_b is the basic concrete breakout strength value for a single fastener in tension in cracked concrete. A_N is the projected area of the failure surface for the fastener or group of fasteners that shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the fastener, or in the case of a group of fasteners, from a line through a row of adjacent fasteners. A_N shall not exceed nA_{No}, where n is the number of tensioned fasteners in the group. A_{No} is the projected area of the failure surface of a single fastener remote from edges:

$$A_{No} = 9h_{ef}^2 \quad \text{(A-6)}$$

A.5.2.2 – Unless determined in accordance with A.4.2, the basic concrete breakout strength of a single fastener in tension in cracked concrete shall not exceed:

$$N_b = k\sqrt{f'_c}\,h_{ef}^{1.5} \quad \text{(A-7a)}$$

where k =24

Alternatively, for cast-in headed studs and headed bolts with 11 in. $\le h_{ef} \le$ 25 in., the basic concrete breakout strength of a single fastener in tension in cracked concrete shall not exceed:

$$N_b = k\sqrt{f'_c}\,h_{ef}^{5/3} \quad \text{(A-7b)}$$

where k = 16.

A.5.2.3 – For the special case of fasteners in an application with three or four edges and the largest edge distance $c_{max} \leq 1.5h_{ef}$, the embedment depth h_{ef} used in Eq. (A-6), (A-7), (A-8) and (A-9) shall be limited to $c_{max}/1.5$.

A.5.2.4 – The modification factor for eccentrically loaded fastener groups is:

$$\psi_1 = \frac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)} \leq 1 \qquad \text{(A-8)}$$

Eq. (A-8) is valid for $e_N' \leq s/2$.

If the loading on a fastener group is such that only some fasteners are in tension, only those fasteners that are in tension shall be considered when determining the eccentricity, e_N', for use in Eq. (A-8).

In the case where eccentric loading exists about two axes, the modification factor, ψ_1, shall be computed for each axis individually and the product of these factors used as ψ_1 in Eq. (A-5).

A.5.2.5 – The modification factor for edge effects is:

$$\psi_2 = 1 \text{ if } c_{min} \geq 1.5h_{ef} \qquad \text{(A-9a)}$$

$$\psi_2 = 0.7 + 0.3\frac{c_{min}}{1.5h_{ef}} \text{ if } c_{min} < 1.5h_{ef} \qquad \text{(A-9b)}$$

A.5.2.6 – When a fastener is located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ at service load levels, the following modification factor shall be permitted:

$$\psi_3 = 1.25$$

A.5.2.7 – When analysis indicates cracking at service load levels, ψ_3 shall be taken as 1.0. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with 10.6.4*, or equivalent crack control shall be provided by confining reinforcement.

A.5.2.8 – When an additional plate or washer is added at the head of the fastener, it shall be permitted to calculate the projected area of the failure surface by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than t from the outer edge of the head of fastener, where t is the thickness of the washer or plate.

A.5. 3 — Pullout strength of fastener in tension

A.5.3.1 – Unless determined in accordance with A.4.2, the nominal pullout strength of a fastener in tension shall not exceed:

$$N_{pn} = \psi_4 N_P \qquad \text{(A-10)}$$

A.5.3.2 – For single cast-in headed studs and headed bolts, it shall be permitted to evaluate the pullout strength in tension using A.5.3.3. For single J-bolts or L-bolts, it shall be permitted to evaluate the pullout strength in tension using A.5.3.4. Alternatively, it shall be permitted to use values of N_p based on the 5 percent fractile of tensile tests performed in such a manner as to exclude the benefit of friction.

A.5.3.3 – Unless determined in accordance with A.4.2, the pullout strength in tension of a single headed stud or headed bolt, N_p for use in Eq. (A-10), shall not exceed:

$$N_p = A_b \, 8 \, f'_c \qquad \text{(A-11a)}$$

**See ACI 318-99.*

A.5.3.4 – Unless determined in accordance with A.4.2 the pullout strength in tension of a single J-bolt or L-bolt, N_p for use in Eq. (A-10) shall not exceed:

$$N_p = 0.9 f'_c e_h d_o \qquad \text{(A-11b)}$$

where $3d_o \leq e_h \leq 4.5d_o$.

A.5.3.5 – For a fastener located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ at service load levels, the following modification factor shall be permitted:

$$\psi_4 = 1.4$$

Otherwise, ψ_4 shall be taken as 1.0.

A.5.4 – Concrete side-face blowout strength of a headed fastener in tension

A.5.4.1 – For a single headed fastener with deep embedment close to an edge $(c < 0.4\, h_{ef})$, unless determined in accordance with A.4.2, the nominal side-face blowout strength N_{sb} shall not exceed:

$$N_{sb} = 160c\sqrt{A_b}\sqrt{f'_c} \qquad \text{(A-12)}$$

If the single fastener is located at a perpendicular distance c_2 less than $3c$ from an edge, the value of N_{sb} shall be modified by multiplying it by the factor $(1 + c_2/c)/4$ where $1 \leq c_2/c \leq 3$.

A.5.4.2 – For multiple headed fasteners with deep embedment close to an edge $(c < 0.4\, h_{ef})$ and spacing between fasteners less than $6c$, unless determined in accordance with A.4.2, the nominal strength of the group of fasteners for a side-face blowout failure N_{sbg} shall not exceed:

$$N_{sbg} = \left(1 + \frac{s_o}{6c}\right)N_{sb} \qquad \text{(A-13)}$$

where s_o = spacing of the outer fasteners along the edge in the group and N_{sb} is obtained from Eq. (A-12) without modification for a perpendicular edge distance.

A.6 – Design requirements for shear loading

A.6.1 – Steel strength of fastener in shear

A.6.1.1 – The nominal strength of a fastener in shear as governed by steel, V_s, shall be evaluated by calculations based on the properties of the fastener material and the physical dimensions of the fastener. Alternatively, it shall be permitted to use values based on the 5 percent fractile of test results to establish values of V_s.

A.6.1.2 – Unless determined by the 5 percent fractile of test results, nominal strength of a fastener or group of fasteners in shear shall not exceed:

(a) for cast-in fasteners with a well defined yield point:

$$V_s = nA_{se} f_y \qquad \text{(A-14)}$$

(b) for cast-in fasteners without a well defined yield point:

$$V_s = n0.6A_{se} f_{ut} \qquad \text{(A-15)}$$

where f_{ut} shall not be taken greater than 125,000 psi.

A.6.1.3 – Where fasteners are used with built-up grout pads, the nominal strengths of A.6.1.2 shall be reduced by 20 percent.

A.6.2 – Concrete breakout strength of fastener in shear

A.6.2.1 – Unless determined in accordance with A.4.2, nominal concrete breakout strength in shear of a fastener or group of fasteners shall not exceed:

(a) for shear force perpendicular to the edge on a single fastener:

$$V_{cb} = \frac{A_V}{A_{Vo}} \psi_6 \psi_7 V_b \qquad \text{(A-16a)}$$

(b) for shear force perpendicular to the edge on a group of fasteners:

$$V_{cbg} = \frac{A_V}{A_{Vo}} \psi_5 \psi_6 \psi_7 V_b \qquad \text{(A-16b)}$$

(c) for shear force parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value for shear force determined from Eq. (A-16a or b), respectively, with ψ_6 taken equal to **1**.

(d) For fasteners located at a corner, the limiting nominal concrete breakout strength shall be determined for each edge and the minimum value shall be used.

V_b is the basic concrete breakout strength value for a single fastener. A_V is the projected area of the failure surface on the side of the concrete member at its edge for a single fastener or a group of fasteners. It shall be permitted to evaluate this area as the base of a truncated half pyramid projected on the side face of the member where the top of the half pyramid is given by the axis of the fastener row selected as critical. The value of c_1 shall be taken as the distance from the edge to this axis. A_V shall not exceed nA_{Vo}, where n is the number of fasteners in the group.

A_{Vo} is the projected area for a single fastener in a deep member and remote from edges in the direction perpendicular to the shear force. It shall be permitted to evaluate this area as the base of a half pyramid with a side length parallel to the edge of $3c_1$ and a depth of $1.5c_1$:

$$A_{Vo} = 4.5c_1^2 \qquad \text{(A-17)}$$

Where fasteners are located at varying distances from the edge and the fasteners are welded to the attachment so as to distribute the force to all fasteners, it shall be permitted to evaluate the strength based on the distance to the farthest row of fasteners from the edge. In this case, it shall be permitted to base the value of c_1 on the distance from the edge to the axis of the farthest fastener row which is selected as critical, and all of the shear shall be assumed to be carried by this critical fastener row alone.

A.6.2.2 – Unless determined in accordance with A.4.2, the basic concrete breakout strength in shear of a single fastener in cracked concrete shall not exceed:

$$V_b = 7\left(\frac{\ell}{d_o}\right)^{0.2} \sqrt{d_o}\,\sqrt{f_c'}\,c_1^{1.5} \qquad \text{(A-18a)}$$

A.6.2.3 – For cast-in headed studs, headed bolts, or hooked bolts, that are rigidly welded to steel attachments having a minimum thickness equal to the greater of 3/8 in. or half of the fastener diameter, unless determined in accordance with A.4.2, the basic concrete breakout strength in shear of a single fastener in cracked concrete shall not exceed:

$$V_b = 8\left(\frac{\ell}{d_o}\right)^{0.2} \sqrt{d_o}\,\sqrt{f'_c}\,c_1^{1.5} \qquad \text{(A-18b)}$$

provided that:

a) for groups of fasteners, the strength is determined based on the strength of the row of fasteners farthest from the edge

b) the center-to-center spacing of the fasteners is not less than 2.5 in.

c) supplementary reinforcement is provided at the corners if $c_2 \le 1.5h_{ef}$.

A.6.2.4 – For the special case of fasteners in a thin member influenced by three or more edges, the edge distance c_1 used in Eq. (A-17), (A-18), (A-19) and (A-20) shall be limited to $h/1.5$.

A.6.2.5 – The modification factor for eccentrically loaded fastener groups is:

$$\psi_5 = \frac{1}{1+\dfrac{2e'_v}{3c_1}} \le 1 \qquad \text{(A-19)}$$

Eq. (A-19) is valid for $e'_v \le s/2$.

A.6.2.6 – The modification factor for edge effects is:

$$\psi_6 = 1 \text{ if } c_2 \ge 1.5c_1 \qquad \text{(A-20a)}$$

$$\psi_6 = 0.7 + 0.3\frac{c_2}{1.5c_1} \text{ if } c_2 < 1.5c_1 \qquad \text{(A-20b)}$$

A.6.2.7 – For fasteners located in a region of a concrete member where analysis indicates no cracking $(f_t < f_r)$ at service loads, the following modification factor shall be permitted:

$$\psi_7 = 1.4$$

For fasteners located in a region of a concrete member where analysis indicates cracking at service load levels, the following modification factors shall be permitted. In order to be considered as edge reinforcement, the reinforcement shall be designed to intersect the concrete breakout:

$\psi_7 = 1.0$ for fasteners in cracked concrete with no edge reinforcement or edge reinforcement smaller than a No. 4 bar

$\psi_7 = 1.2$ for fasteners in cracked concrete with edge reinforcement of a No. 4 bar or greater between the fastener and the edge

$\psi_7 = 1.4$ for fasteners in cracked concrete with edge reinforcement of a No. 4 bar or greater between the fastener and the edge and with the edge reinforcement enclosed within stirrups spaced at not more than 4 in.

A.6.3 — Concrete pryout strength of fastener in shear

A.6.3.1 – Unless determined in accordance with A.4.2, the nominal pryout strength, V_{cp}, shall not exceed:

$$V_{cp} = k_{cp}N_{cb} \qquad \text{(A-21)}$$

where

k_{cp} = 1.0 for h_{ef} < 2.5 in.
k_{cp} = 2.0 for $h_{ef} \geq$ 2.5 in.
and N_{cb} shall be determined from Eq. (A-5a), lb.

A.7 — Interaction of tensile and shear forces

Unless determined in accordance with A.4.3, fasteners or groups of fasteners that are subjected to both shear and axial loads shall be designed to satisfy the requirements of A.7.1 through A.7.3. The value of ϕN_n shall be the smallest of the steel strength of the fastener in tension, concrete breakout strength of fastener in tension, pullout strength of fastener in tension, and side-face blowout strength. The value of ϕV_n shall be the smallest of the steel strength of fastener in shear, the concrete breakout strength of fastener in shear, and the pryout strength.

A.7.1 – If $V_u \leq 0.2\phi V_n$, then full strength in tension shall be permitted: $\phi N_n \geq N_u$.

A.7.2 – If $N_u \leq 0.2\phi N_n$, then full strength in shear shall be permitted: $\phi V_n \geq V_u$.

A.7.3 – If $V_u > 0.2\phi V_n$ and $N_u > 0.2\,\phi\,N_n$, then:

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \leq 1.2 \qquad \text{(A-22)}$$

A.8 — Required edge distances and spacings to preclude splitting failure

Minimum spacings and edge distances for fasteners shall conform to A.8.1 and A.8.2, unless reinforcement is provided to control splitting.

A.8.1 – Unless determined in accordance with A.8.2, minimum edge distances for cast-in headed fasteners that will not be torqued shall be based on minimum cover requirements for reinforcement in 7.7. For cast-in headed fasteners that will be torqued, the minimum edge distances shall be $6d_o$.

A.8.2 – For fasteners that will remain untorqued, if the edge distance or spacing is less than those specified in A.8.1, calculations shall be performed using a fictitious value of d_o that meets the requirements of A.8.1. Calculated forces applied to the fastener shall be limited to the values corresponding to a fastener having that fictitious diameter.

A.8.3 – Project drawings and project specifications shall specify use of fasteners with a minimum edge distance as assumed in design.

A.9 – Installation of fasteners

A.9.1 – Fasteners shall be installed in accordance with the project drawings and project specifications.

Appendix B
Commentary on Design Provisions

B.0 — Notation

A_{se} For threaded bolts, ANSI/ASME B1.1[B.1] defines A_{se} as:

$$A_{se} = \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2$$

where n_t is the number of threads per inch.

e_n Actual eccentricity of a normal force on an attachment

h_{ef} Effective embedment depths for a variety of fastener types are shown in Fig. B.1.

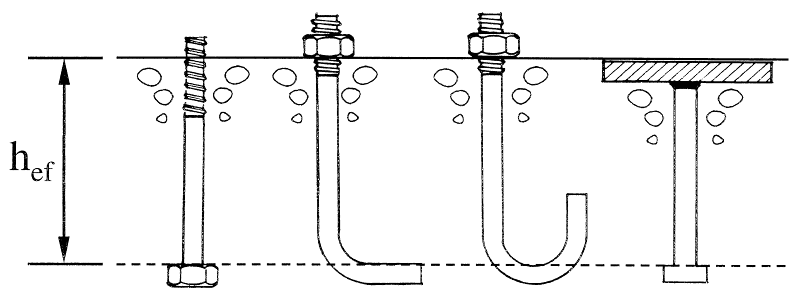

Figure B.1 Types of cast-in-place fasteners

B.1 — Definitions

5 Percent Fractile – The determination of the coefficient K associated with the 5 percent fractile, $\bar{x}$ - $K\sigma$, depends on the number of tests, n, used to compute $\bar{x}$ and σ. Values of K range, for example, from 1.645 for $n = \infty$ to 2.010 for $n = 40$ and 2.568 for $n = 10$.

B.2 — Scope

B.2.1 – Appendix A is restricted in scope to structural fasteners that transmit structural loads from attachments into concrete members. The levels of safety defined by the combinations of load factors and ϕ factors are appropriate for structural applications. Other standards can require more stringent safety levels during temporary handling.

B.2.2 – The wide variety of shapes and configurations of specialty inserts makes it difficult to prescribe generalized tests and design equations for many insert types. Hence, they have been excluded from the scope of Appendix A. Post-installed mechanical and adhesive fasteners are widely used and can perform adequately. However, at this time such fasteners are outside the scope of this appendix.

B.2.3 – Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1[B.1], B18.2.1[B.2], and B18.2.6[B.3] have been tested and have proven to behave predictably, so calculated pullout values are acceptable.

B.2.4 – The exclusion from the scope of load applications producing high-cycle fatigue or extremely short duration impact (such as blast or shock wave) are not meant to exclude seismic load effects. B.3.3 presents additional requirements for design when seismic loads are included.

B.3 — General requirements

B.3.1 – When the strength of a fastener group is governed by breakage of the concrete, the behavior is brittle and there is limited redistribution of the forces between the highly stressed and less stressed fasteners. In this case, the theory of elasticity is required to be used assuming the attachment that distributes loads to the fasteners is sufficiently stiff. The forces in the fasteners are considered to be proportional to the external load and its distance from the neutral axis of the fastener group.

If fastener strength is governed by ductile yielding of the fastener steel, significant redistribution of fastener forces can occur. In this case, an analysis assuming the theory of elasticity will be conservative. References B.4 to B.6 discuss non-linear analysis, using theory of plasticity, for the determination of the capacities of ductile fastener groups.

B.3.3 – The design of fasteners in zones of moderate or high seismic risk is based on a more conservative approach by the introduction of a 0.75 factor on the design strength ϕN_n and ϕV_n, and by requiring ductile failures. Alternatively, a higher value of fastener strength can be used if the attachment being fastened is designed to ensure ductile yielding of the attachment at a load well below the minimum probable fastener strength.

For a fastener to be acceptable in seismic loading situations the system is required to have adequate ductility. If the fastener cannot meet these requirements, then the attachment is required to be designed so as to yield at a load well below the fastener capacity. In designing attachments for adequate ductility the ratio of yield to ultimate load capacity should be considered. A connection element could yield only to result in a secondary failure as one or more elements strain harden if the ultimate load capacity is excessive when compared to the yield capacity.

Under seismic conditions, the direction of shear loading may not be predictable. The full shear load should be assumed in any direction for a safe design.

B.3.5 – A limited number of tests of cast-in fasteners in high-strength concrete[B.7] indicate that the design procedures contained in this Appendix become unconservative at f'_c = 11,000 to 12,000 psi. Until further tests are available, an upper limit of f'_c = 10,000 psi was imposed in the design of cast-in fasteners. This is consistent with Chapters 11 and 12*.

B.4 — General requirements for strength of structural fasteners

B.4.1 – This section provides the requirements for establishing the strength of fasteners to concrete. The various types of steel and concrete failure modes for fasteners are shown in Figs. B.4.1A and B.4.1B. Comprehensive discussions of fastener failure modes are included in References B.8 to B.10. Any model that complies with the requirements of A.4.2 and A.4.3 can be used to establish the concrete related strengths. For fasteners such as headed bolts and headed studs, the concrete breakout design method of A.5.2 and A.6.2 is acceptable. The fastener strength is also dependent on the pullout strength of A.5.3, the side-face blowout strength of A.5.4 and the minimum spacings and edge distances of A.8.

Test procedures can also be used to determine the single-fastener breakout strength in tension and in shear. However, the test results are required to be evaluated on a basis statistically equivalent to that used to select the values for the concrete breakout method "considered to satisfy" provisions of A.4.2. The basic strength cannot be taken greater than the 5 percent fractile. The number of tests has to be sufficient for statistical validity and should be considered in the determination of the 5 percent fractile.

**See ACI 318-99.*

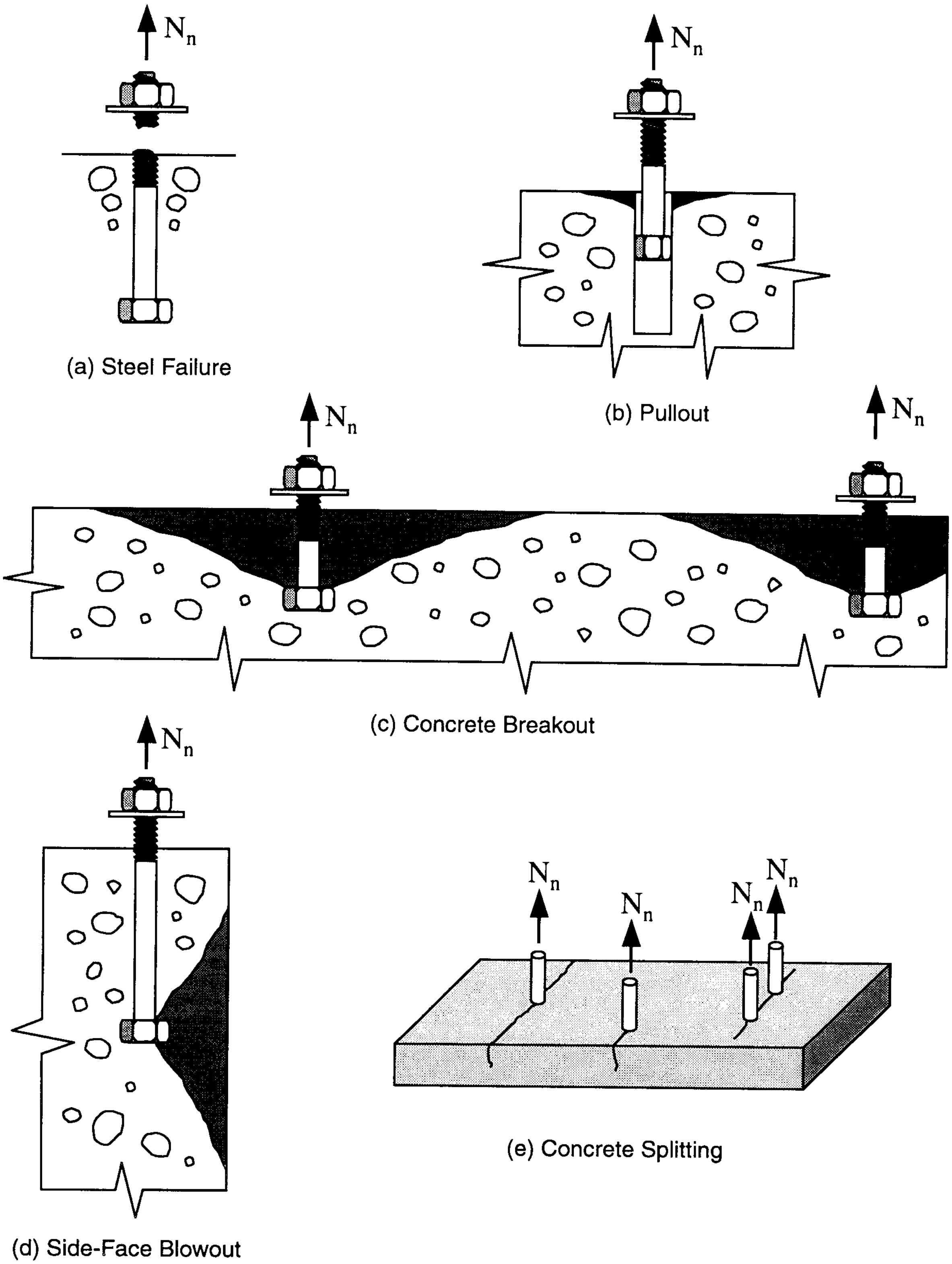

Figure B.4.1A Failure modes for fasteners under tensile loading

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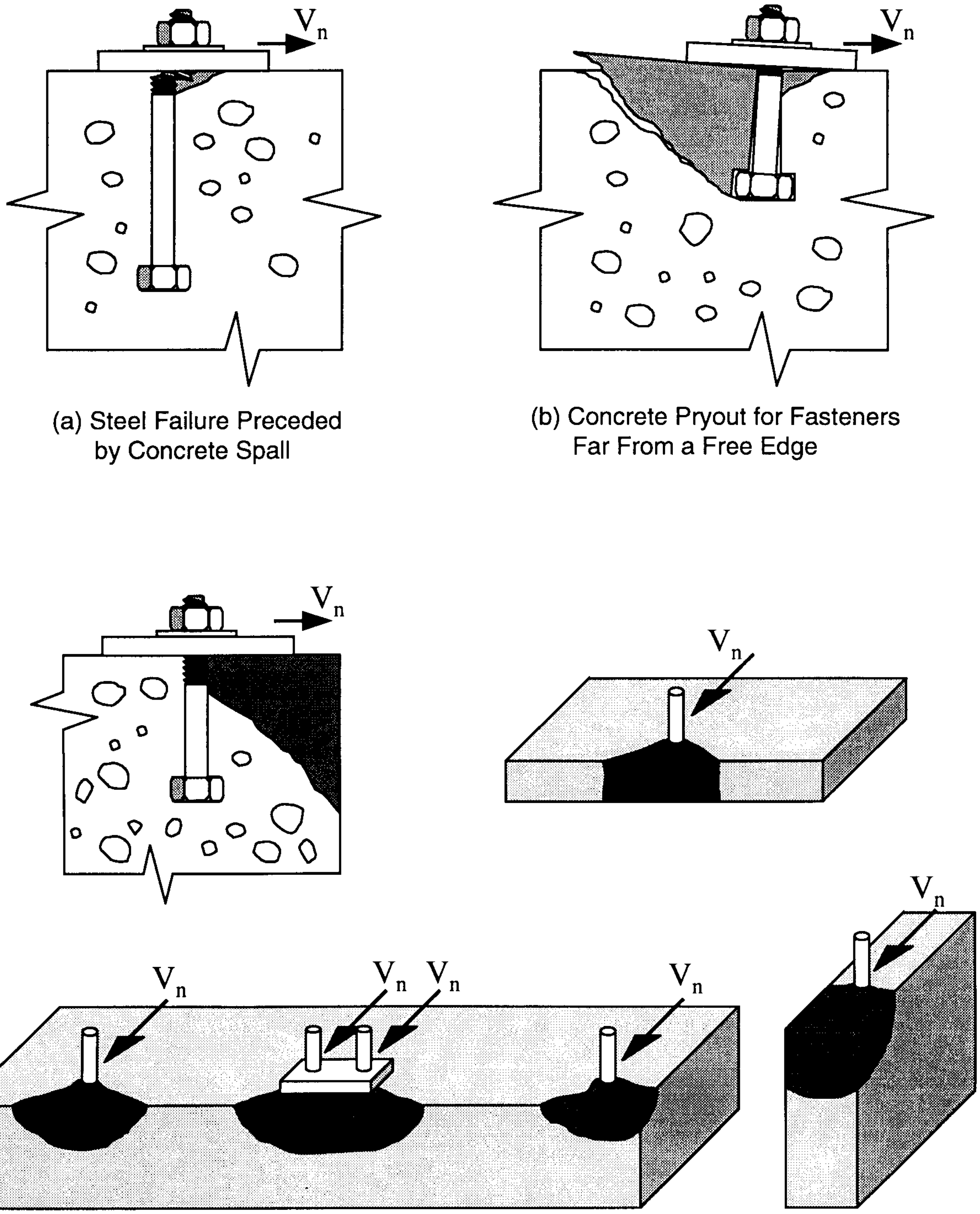


(a) Steel Failure Preceded by Concrete Spall

(b) Concrete Pryout for Fasteners Far From a Free Edge

(c) Concrete Breakout

Figure B.4.1B Failure modes for fasteners under shear loading

B.4.2 and 4.3 – A.4.2 and A.4.3 establish the performance factors for which fastener design models are required to be verified. Many possible design approaches exist and the user is always permitted to "design by test" using A.4.2 as long as sufficient data are available to verify the model.

B.4.2.1 – The addition of supplementary reinforcement in the direction of the load, confining reinforcement, or both, can greatly enhance the strength and ductility of the fastener connection. Such enhancement is practical with cast-in fasteners such as those used in precast sections.

The shear strength of headed fasteners located near the edge of a member can be significantly increased with appropriate supplementary reinforcement. References B.8, B.11 and B.12 provide substantial information on design of such reinforcement. The effect of such supplementary reinforcement is not included in the concrete breakout calculation method of A.5.2 and A.6.2. The designer has to rely on other test data and design theories in order to include the effects of supplementary reinforcement.

The breakout strength of an unreinforced connection can be taken as an indication of the load at which significant cracking will occur. Such cracking can represent a serviceability problem if not controlled. (See B.6.2.1)

B.4.2.2 – The method for concrete breakout design included as "considered to satisfy" A.4.2 was developed from the Concrete Capacity Design (CCD) Method[B.9,B.10], which was an adaptation of the κ Method[B.13,B.14] and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the load-bearing capacity of a fastener or group of fasteners by using a basic equation for tension or for shear for a single fastener in cracked concrete, and multiplying by factors which account for the number of fasteners, edge distance, spacing, eccentricity and absence of cracking. The limitations on fastener size and embedment length are based on the current range of test data.

The breakout strength calculations are based on a model suggested in the κ Method. It is consistent with a breakout prism angle of approximately 35 deg (Fig. B.4.2(a) and (b)).

B.4.4 – The ϕ factor for failure of ductile elements is indicative of less variability in steel tension failures than concrete breakout failures, and the greater amount of warning with a ductile failure. It is acceptable to have a ductile failure of a steel element in the attachment if the attachment is designed so that it will undergo ductile yielding at a load level no greater than 75 percent of the minimum fastener design strength (See A.3.3.3). For fasteners governed by the more brittle concrete breakout or blowout failure, two conditions are recognized. If supplementary reinforcement is provided to tie the failure prism into the structural member (Condition A), more ductility is present than in the case where such supplementary reinforcement is not present (Condition B). Design of supplementary reinforcement is discussed in B4.2.1 and References B.4-B.6 and B.17. Even though the ϕ factor for plain concrete uses a value of 0.65, the basic factor for brittle failures ($\phi = 0.75$) has been chosen based on the results of probabilistic studies[B.15] that indicated that for fastening to concrete the use of $\phi = 0.65$ with <u>mean</u> values of concrete-controlled failures produced adequate safety levels. However, the nominal resistance expressions used in this appendix and in the test requirements are the 5 percent fractiles. Thus, the $\phi = 0.65$ value would be overly conservative. Comparison with other design procedures and probabilistic studies[B.15] indicated that the choice of $\phi = 0.75$ was justified. For applications with supplementary reinforcement and more ductile failures (Condition A), the ϕ factors are increased. The value of $\phi = 0.85$ is compatible with the level of safety for shear failures in concrete beams, and has been recommended by the PCI Design Handbook[B.16] and ACI 349[B.17].

B4.5 – The ϕ factors for use with the Load Factors of Appendix C* were determined in a manner consistent with the other ϕ factors of Appendix C.

**See ACI 318-99.*

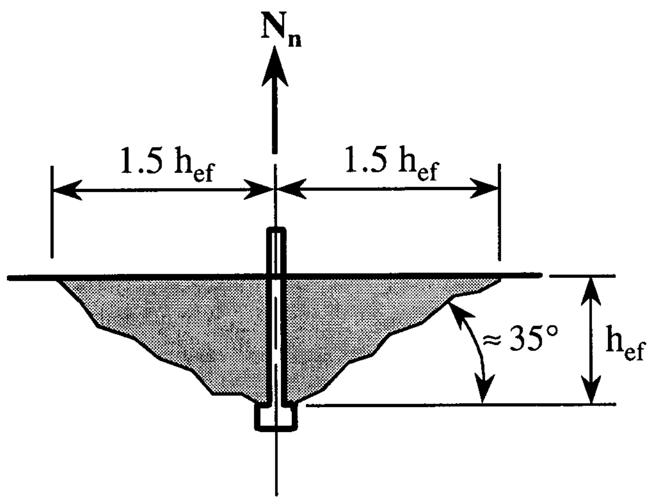


Figure B.4.2(a) Breakout cone for tension

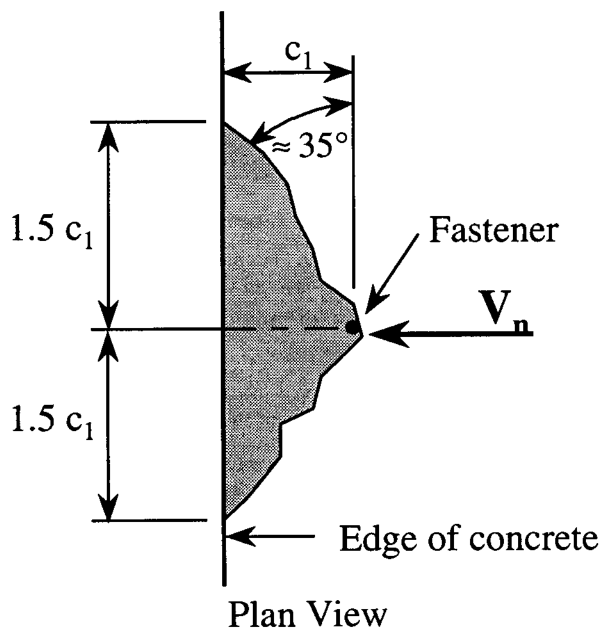


Figure B.4.2(b) Breakout cone for shear

B.5 — Design requirements for tensile loading

B.5.2 — Concrete breakout strength of fastener in tension

B.5.2.1 – The effects of multiple fasteners, spacing of fasteners, and edge distance on the nominal concrete breakout strength in tension are included by applying the modification factors $\boldsymbol{A_N / A_{No}}$ and ψ_2 in Eq. A-5.

Figure B.5.1(a) shows $\boldsymbol{A_{No}}$ and the development of Eq. (A-6). $\boldsymbol{A_{No}}$ is the maximum projected area for a single fastener. Figure B.5.1(b) shows examples of the projected areas for various single fastener and multiple fastener arrangements. Because $\boldsymbol{A_N}$ is the total projected area for a group of fasteners, and $\boldsymbol{A_{No}}$ is the area for a single fastener, there is no need to include $\boldsymbol{n}$, the number of fasteners, in Eq. (A-5a) or (A-5b). If fastener groups are positioned in such a way that their projected areas overlap, the value of $\boldsymbol{A_N}$ is required to be reduced accordingly.

B.5.2.2 – The basic equation for fastener capacity was derived[B.9,B.10,B.11,B.14] assuming a concrete failure prism with an angle of about 35 deg, and considering fracture mechanics concepts. The values of $\boldsymbol{k}$ were determined from a large database of test results in uncracked concrete[B.9] as the 5 percent fractile. The values were adjusted to corresponding $\boldsymbol{k}$ values for cracked concrete[B.10,B.18] For fasteners with a deep embedment ($\boldsymbol{h_{ef}}$ > 11 in.) some test evidence indicates the use of $\boldsymbol{h_{ef}^{1.5}}$ can be overly conservative for some cases. Often such tests have been with selected aggregates for special applications. An alternate expression [Eq. (A-7b)] is provided using $\boldsymbol{h_{ef}^{5/3}}$ for evaluation of cast-in fasteners with 11 in. $\leq \boldsymbol{h_{ef}} \leq$ 25 in. The limit of 25 in. corresponds to the upper range of test data.

B.5.2.3 – For fasteners influenced by three or more edges where any edge distance is less than $\boldsymbol{1.5h_{ef}}$, the tensile breakout strength computed by the ordinary CCD Method, which is the basis for Eq. (A-7), gives misleading results. This occurs because the ordinary definitions of $\boldsymbol{A_N/A_{No}}$ do not correctly reflect the edge effects. However, if the value of $\boldsymbol{h_{ef}}$ is limited to $\boldsymbol{c_{max}}/1.5$, where $\boldsymbol{c_{max}}$ is the largest of the influencing edge distances that are less than or equal to the actual $\boldsymbol{1.5h_{ef}}$, this problem is corrected. As shown by Lutz[B.19], this limiting value of $\boldsymbol{h_{ef}}$ is to be used in Eqs. (A-6), (A-7), (A-8) and (A-9). This approach is best understood when applied to an actual case. Fig. B.5.2 (a) shows how the failure surface has the same area for any embedment beyond the proposed limit on $\boldsymbol{h_{ef}}$ (taken as $\boldsymbol{h'_{ef}}$ in the figure). In this example, the proposed limit on the value of $\boldsymbol{h_{ef}}$ to be used in the computations where $\boldsymbol{h_{ef}} = \boldsymbol{c_{max}}/1.5$, results in $\boldsymbol{h_{ef}} = \boldsymbol{h'_{ef}}$ = 4 in./1.5 = 2.67 in. This would be the proper value to be used for $\boldsymbol{h_{ef}}$ in computing the resistance, for this example, even if the actual embedment depth is larger.

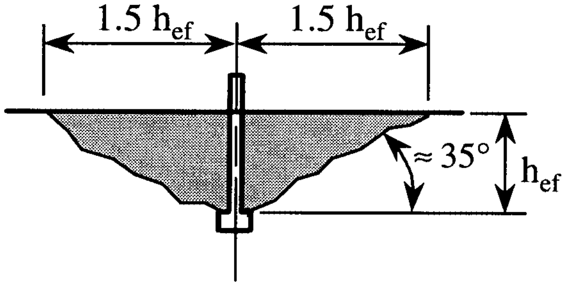
B.5.2.4 – Fig. B.5.2(b) shows dimension $e'_N = e_N$ for a group of fasteners that are all in tension but that have a resultant force eccentric with respect to the centroid of the fastener group. Groups of fasteners can be loaded in such a way that only some of the fasteners are in tension (Fig. B.5.2(c)). In this case, only the fasteners in tension are to be considered in the determination of e'_N. The fastener loading has to be determined as the resultant fastener tension at an eccentricity with respect to the center of gravity of the fasteners in tension.
Eq. (A-8) is limited to cases where $e'_N < s/2$ to alert the designer that all fasteners may not be in tension.

B.5.2.5 – If fasteners are located close to an edge so that there is not enough space for a complete breakout prism to develop, the load bearing capacity of the fastener is further reduced beyond that reflected in $\boldsymbol{A_N/A_{No}}$. If the smallest side cover distance is greater than $\boldsymbol{1.5h_{ef}}$, a complete prism can form and there is no reduction ($\Psi_2 = 1$). If the side cover is less than $\boldsymbol{1.5h_{ef}}$, the factor, Ψ_2, is required to adjust for the edge effect[B.7].

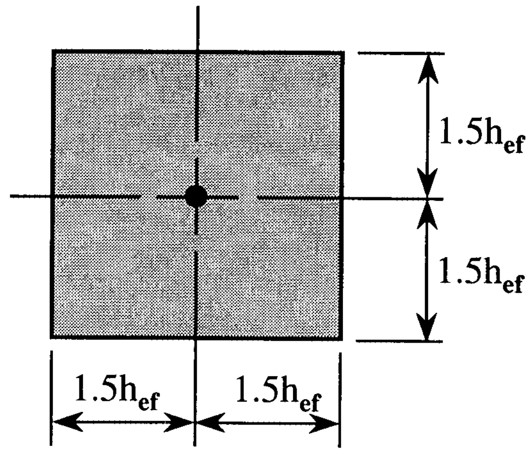
B.5.2.6 – The analysis for the determination of crack formation should include the effects of restrained shrinkage.

B.5.2.7 – If cracks wider than 0.012 in. are expected, confining reinforcement to control the crack width to about 0.012 in. should be provided.

The critical edge distance for headed studs and headed bolts is $1.5h_{ef}$.



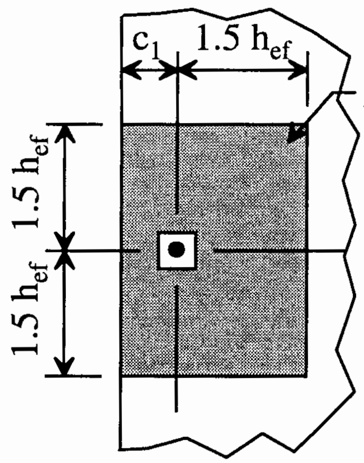
Section through failure cone



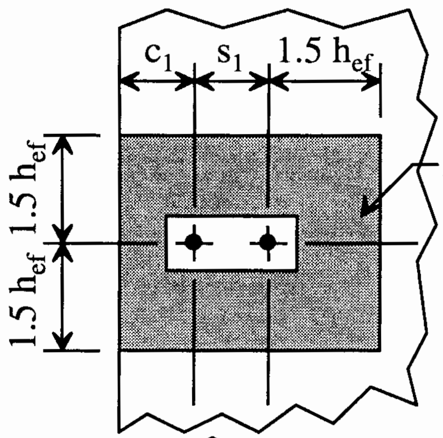
Plan view

$$
\begin{aligned}
A_{No} &= 2 * 1.5h_{ef} \times 2 * 1.5h_{ef} \\
&= 3h_{ef} \times 3h_{ef} \\
&= 9h_{ef}^2
\end{aligned}
$$

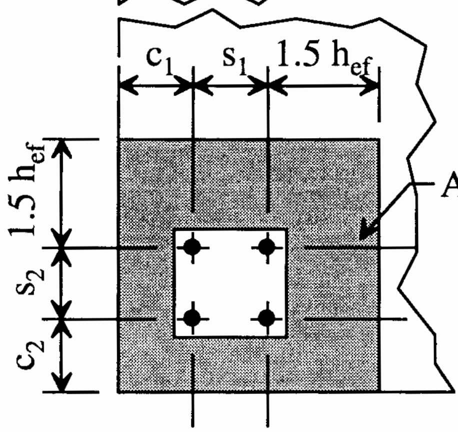
Figure B.5.1(a) Calculation of A_{No}



$$A_N = (c_1 + 1.5h_{ef})\,(2 \times 1.5h_{ef})$$
if $c_1 < 1.5h_{ef}$



$$A_N = (c_1 + s_1 + 1.5h_{ef})\,(2 \times 1.5h_{ef})$$
if $c_1 < 1.5h_{ef}$ and $s_1 < 3h_{ef}$



$$A_N = (c_1 + s_1 + 1.5h_{ef})\,(c_2 + s_2 + 1.5h_{ef})$$
if c_1 and $c_2 < 1.5h_{ef}$
and s_1 and $s_2 < 3h_{ef}$

Figure B.5.1(b) Projected areas for single fasteners and groups of fasteners

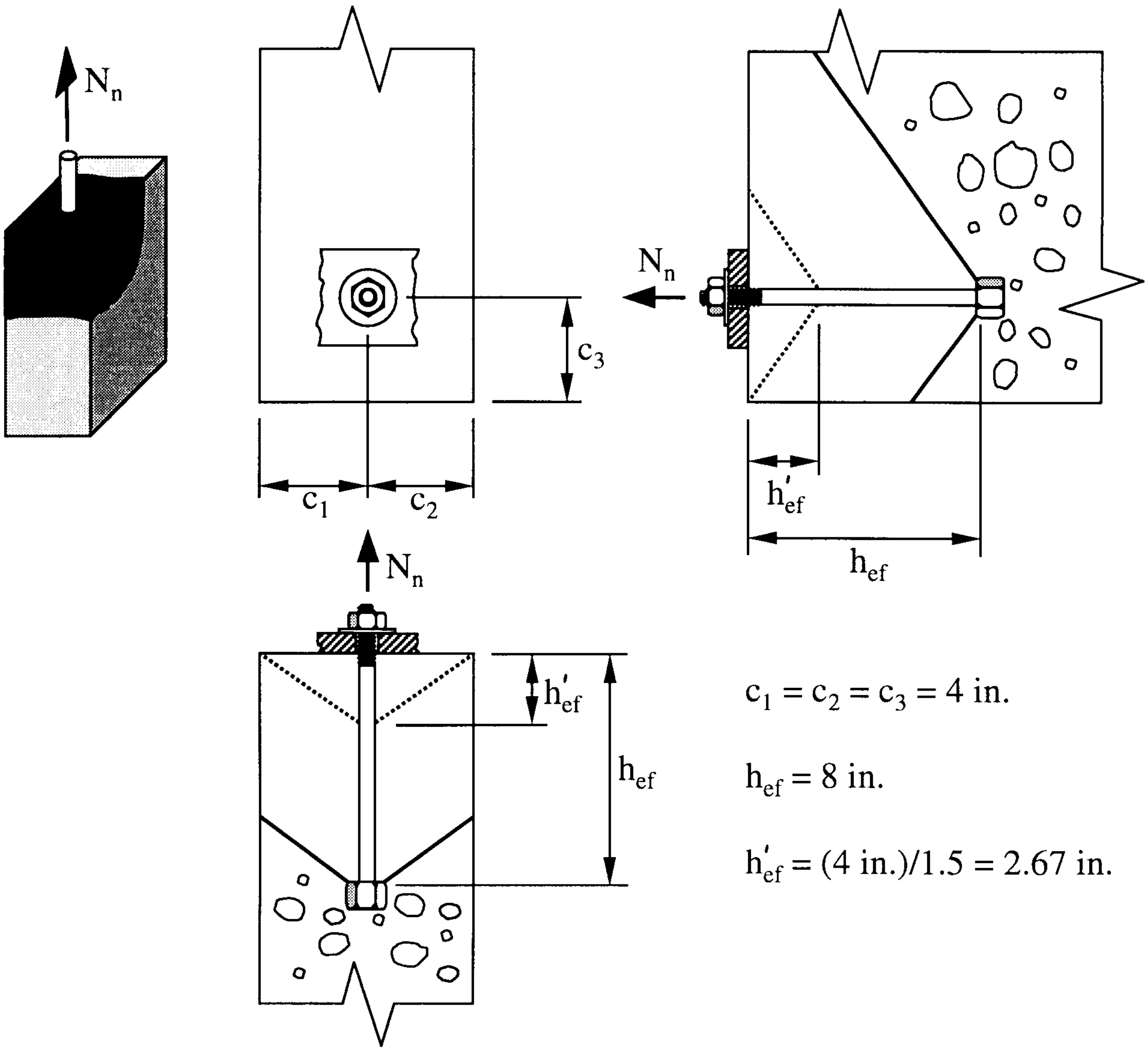


Figure B.5.2(a) Failure surfaces in narrow members for different embedment depths

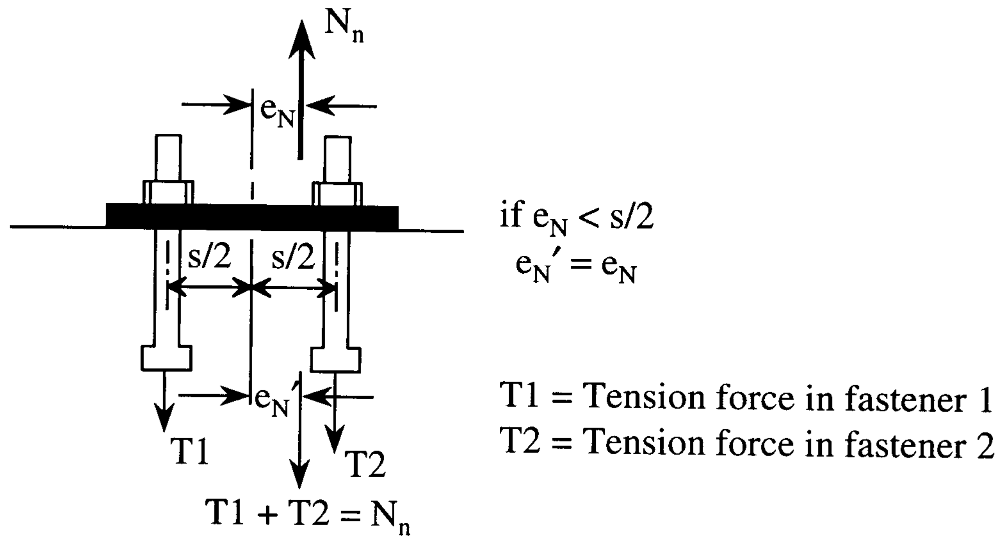


Figure B.5.2(b) Definition of dimension e_N' when all fasteners in a group are in tension

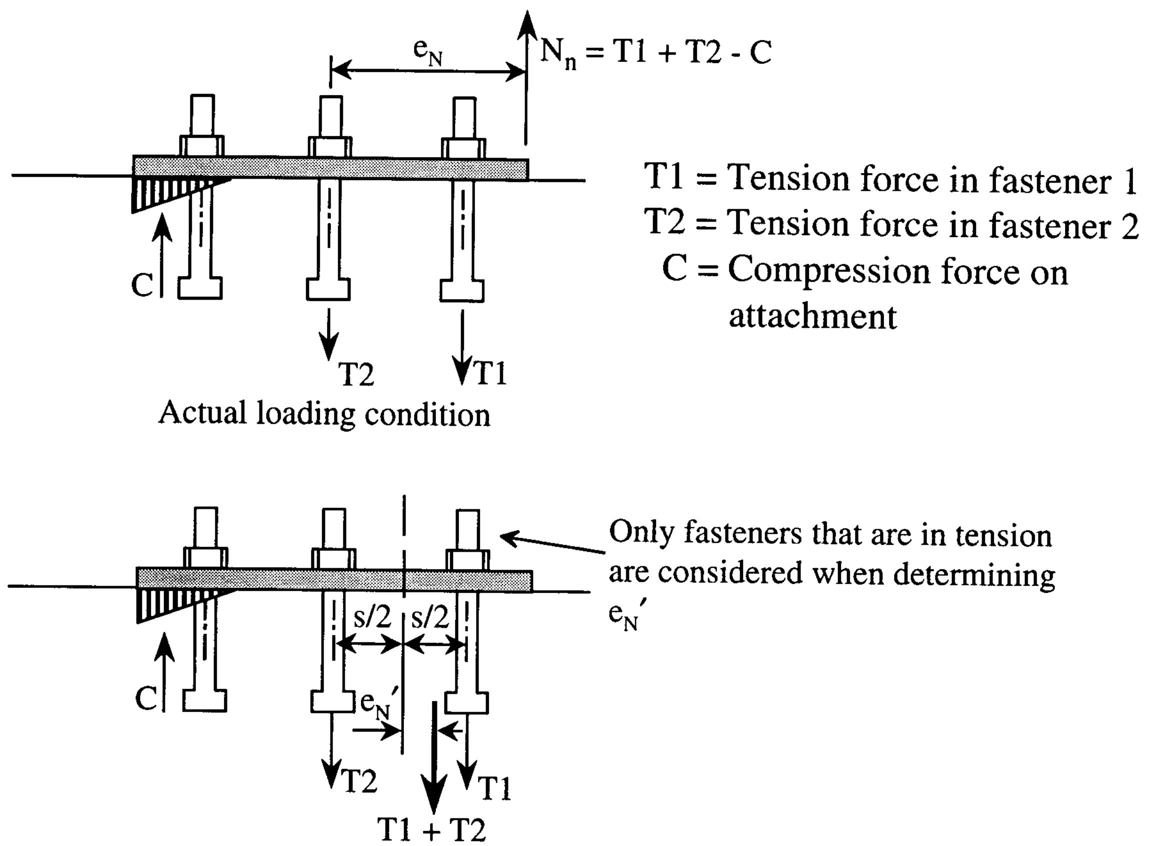


Figure B.5.2(c) Determination of e_N for fastener group with only some fasteners in tension

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B.5.3 — Pullout strength of fastener in tension

B.5.3.2 – The pullout strength in tension of headed studs or headed bolts can be increased by provision of confining reinforcement, such as closely spaced spirals, throughout the head region. This increase can be demonstrated by tests.

B.5.3.3 – Eq. (A-11a) corresponds to the load at which the concrete under the anchor head begins to crush.[B.8,B.17]. It is not the load required to pull the fastener completely out of the concrete, so the equation contains no term relating to embedment depth. The designer should be aware that local crushing under the head will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure.

B.5.3.4 – Eq. (A-11b) for J-bolts and L-bolts was developed by Lutz based on the results of Ref. B.20. Reliance is placed on the bearing component only, neglecting any frictional component since local crushing inside the hook will greatly reduce the stiffness of the connection, and generally will be the beginning of pullout failure.

B.5.4 — Concrete side-face blowout strength of fastener in tension

The design requirements for side-face blowout are based on the recommendations of Ref. B.21. These requirements are applicable to headed fasteners.

B.6 — Design requirements for shear loading

B.6.2 — Concrete breakout strength of fastener in shear

B.6.2.1 – The shear strength equations were developed from the CCD method. They assume a breakout cone angle of approximately 35 deg (Fig. B.4.2(b)), and consider fracture mechanics theory. The effects of multiple fasteners, spacing of fasteners, edge distance and thickness of the concrete member on nominal concrete breakout strength in shear are included by applying the reduction factor $\boldsymbol{A_V/A_{Vo}}$ and ψ_5 in Eq. (A-16). For fasteners far from the edge, B.6.2 usually will not govern. For these cases, A.6.1 and A.6.3 often govern.

Fig. B.6.2(a) shows $\boldsymbol{A_{Vo}}$ and the development of Eq. (A-17). $\boldsymbol{A_{Vo}}$ is the maximum projected area for a single fastener that approximates the surface area of the full breakout prism or cone for a fastener unaffected by edge distance, spacing or depth of member. Fig. B.6.2(b) shows examples of the projected areas for various single fastener and multiple fastener arrangements. $\boldsymbol{A_V}$ approximates the full surface area of the breakout cone for the particular arrangement of fasteners. Since $\boldsymbol{A_V}$ is the total projected area for a group of fasteners, and $\boldsymbol{A_{Vo}}$ is the area for a single fastener, there is no need to include the number of fasteners in the equation.

The assumption shown in Fig. B.6.2(b) with the case for two fasteners perpendicular to the edge is a conservative interpretation of the distribution of the shear force on an elastic basis. If the fasteners are welded to a common plate, when the fastener nearest the front edge begins to form a failure cone, shear load would be transferred to the stiffer and stronger rear fastener. For cases where nominal strength is not controlled by ductile steel elements, Committee 318 has specified in A.3.1 that load effects be determined by elastic analysis. It has been suggested in the PCI Design Handbook approach[B.16] that the increased capacity of the fasteners away from the edge be considered. Because this is a reasonable approach assuming that the fasteners are spaced far enough apart so that the shear failure surfaces do not intersect[B.11], A.6.2 allows such a procedure. If the failure surfaces do not intersect, as would generally occur if the fastener spacing, $\boldsymbol{s}$, is equal to or greater than $\boldsymbol{1.5c_1}$, then after formation of the near-edge failure surface, the higher capacity of the farther fastener would resist most of the load. As shown in the bottom example in Fig. B.6.2(b), it would be appropriate to consider the full shear capacity to be provided by this fastener with its much larger resisting failure surface., No contribution of the fastener near the edge is then considered. It would be advisable to check the near-edge fastener condition to preclude undesirable cracking at service load conditions. Further discussion of design for multiple fasteners is given in Ref. B.8.

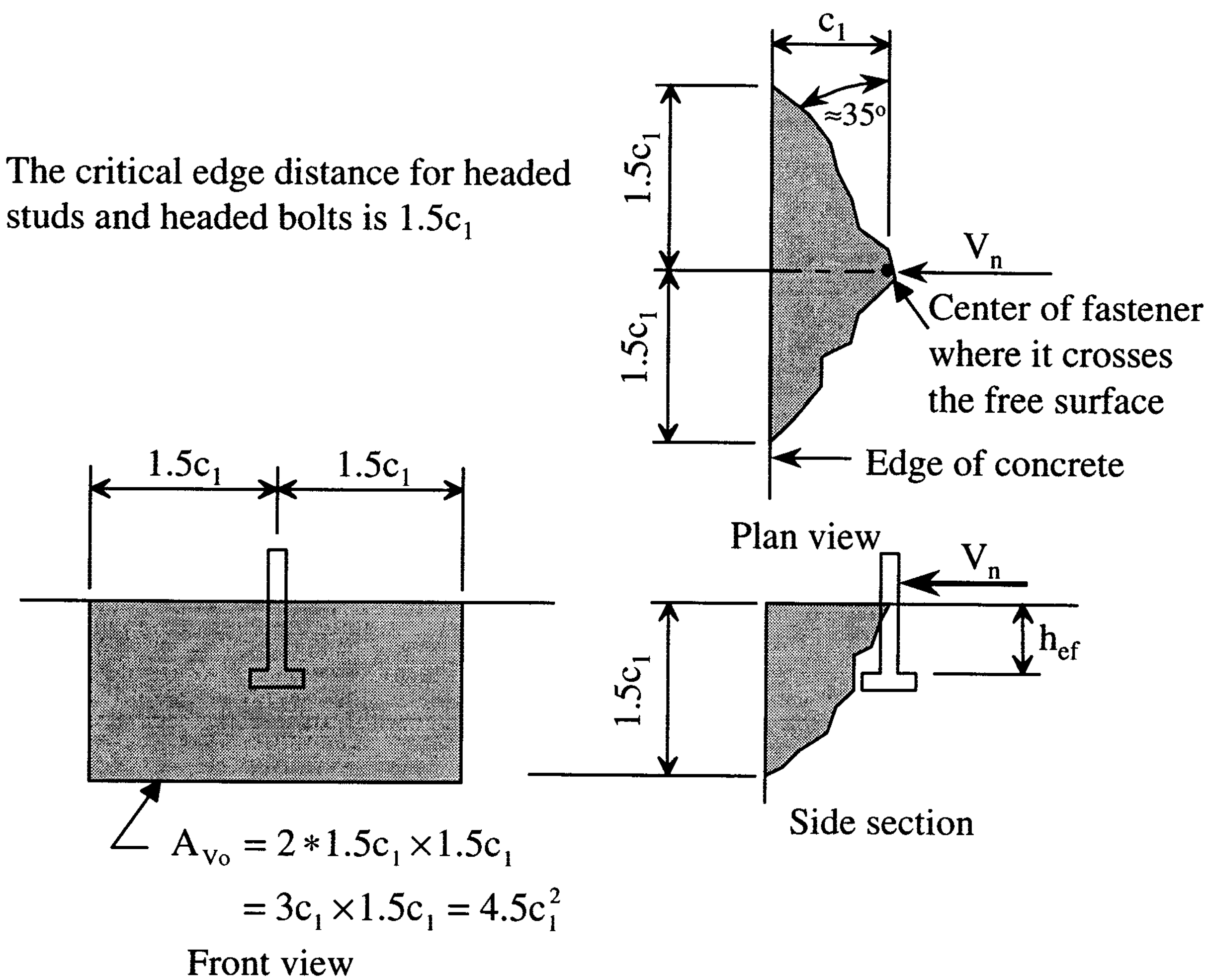
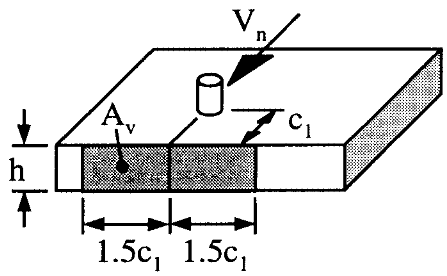


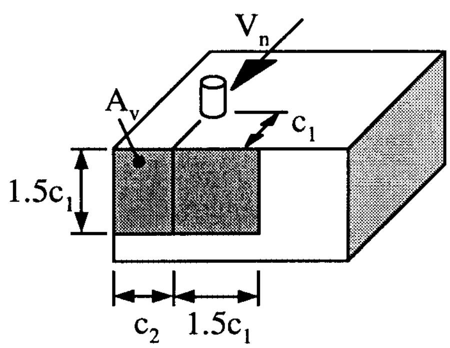
Figure B.6.2(a) Calculation of A_{Vo}

if $h < 1.5c_1$



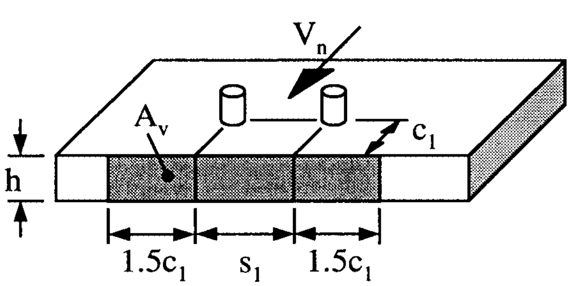
$A_V = 2 \text{ x } 1.5c_1 \text{ x } h$

if $c_2 < 1.5c_1$



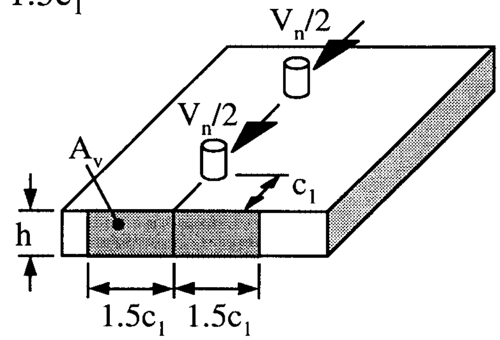
$A_V = 1.5c_1\,(1.5c_1 + c_2)$

if $h < 1.5c_1$ and $s_1 < 3c_1$



$A_V = (2 \text{ x } 1.5c_1 + s_1) \text{ x } h$

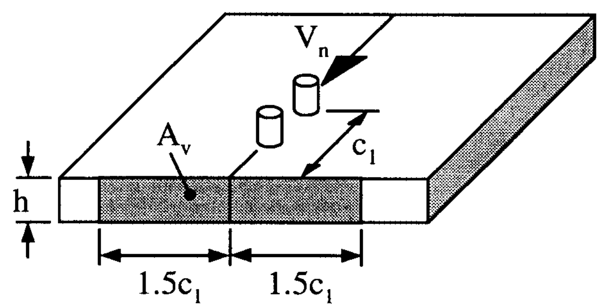
if $h < 1.5c_1$



$A_V = 2 \text{ x } 1.5c_1 \text{ x } h$

Note: One assumption of the distribution of forces indicates that half the shear would be critical on front fastener and its projected area.

if $h < 1.5c_1$



$A_v = 2 \text{ x } 1.5c_1 \text{ x } h$

Note: Another assumption of the distribution of forces that applies only where fasteners are rigidly connected to the attachment indicates that the total shear would be critical on the rear fastener and its projected area.

Figure B.6.2(b) Projected areas for single fasteners and groups of fasteners

For the case of fasteners near a corner subjected to a shear force with components normal to each edge, a satisfactory solution is to independently check the connection for each component of the shear force. Other specialized cases, such as the shear resistance of fastener groups where all fasteners do not have the same edge distance, are treated in Ref. B.11.

The detailed provisions of A.6.2.1 (a) apply to the case of shear force directed towards an edge. When the shear force is directed away from the edge, the strength will usually be governed by A.6.1 or A.6.3.

The case of shear force parallel to an edge (A.6.2.1(c)) is shown in Fig. B.6.2(c). A special case can arise with shear force parallel to the edge near a corner. Take the example of a single fastener near a corner (Fig. B.6.2(d)). If the edge distance to the side c_2 is 40 percent or more of the distance c_1 in the direction of the load, the shear strength parallel to that edge can be computed directly from Eq. (A-16) using c_1 in the direction of the load.

B.6.2.2 – Like the concrete breakout tensile capacity, the concrete breakout shear capacity does not increase with the failure surface, which is proportional to c_1^2. Instead the capacity increases proportionally to $c_1^{1.5}$, due to the size effect. The capacity is also influenced by the fastener stiffness and the fastener diameter[B.9,B.10,B.11,B.14].

The constant 7 in the shear strength equation was determined from test data reported in Ref. B.9 as the 5 percent fractile adjusted for cracking.

B.6.2.3 – For the special case of cast-in headed bolts rigidly welded to an attachment, test data[B.22,B.23] show that somewhat higher shear capacity exists, possibly due to the stiff welding connection clamping the bolt more effectively than an attachment with a fastener gap. Because of this, the basic shear value for such fasteners is increased. Limits are imposed to ensure sufficient rigidity. The design of supplementary reinforcement is discussed in Refs. B.8, B.11 and B.12.

B.6.2.4 – For fasteners influenced by three or more edges where any edge distance is less than $1.5c_1$, the shear breakout strength computed by the basic CCD Method, which is the basis for Eq. (B-18), gives safe but misleading results. These special cases were studied for the κ Method [B.14] and the problem was pointed out by Lutz[B.19]. Similar to the approach used for tensile breakouts in B.5.2.3, a correct evaluation of the capacity is determined if the value of c_1 to be used in Eqs. (A-17), (A-18), (A-19) and (A-20) is limited to $h/1.5$.

B.6.2.5 – This section provides a modification factor for an eccentric shear force towards an edge on a group of fasteners. If the shear load originates above the plane of the concrete surface, the shear should first be resolved as a shear in the plane of the concrete surface, with a moment that can or cannot also cause tension in the fasteners, depending on the normal force. Fig. B.6.2(e) defines the term e'_V for calculating the Ψ_5 modification factor that accounts for the fact that more shear is applied on one fastener than the other, tending to split the concrete near an edge. If $e'_V > s/2$, the CCD procedure is not applicable.

B.6.2.6 – Fig. B.6.2(f) shows the dimension c_2 for the Ψ_6 calculation.

B.6.3 — Concrete pryout strength

B.6.3 – Ref. B.9 indicates that the pryout shear resistance can be approximated as 1 to 2 times the fastener tensile resistance with the lower value appropriate for h_{ef} less than 2.5 in.

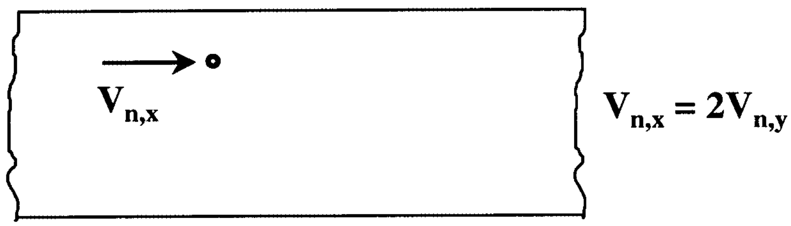


Figure B.6.2(c) Shear force parallel to an edge

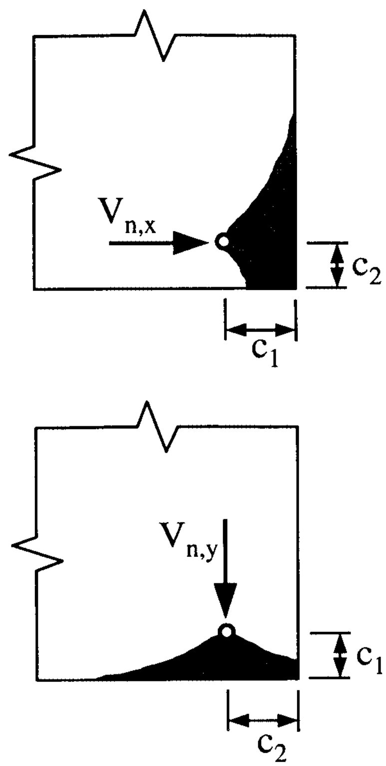


Figure B.6.2(d) Fasteners near a corner

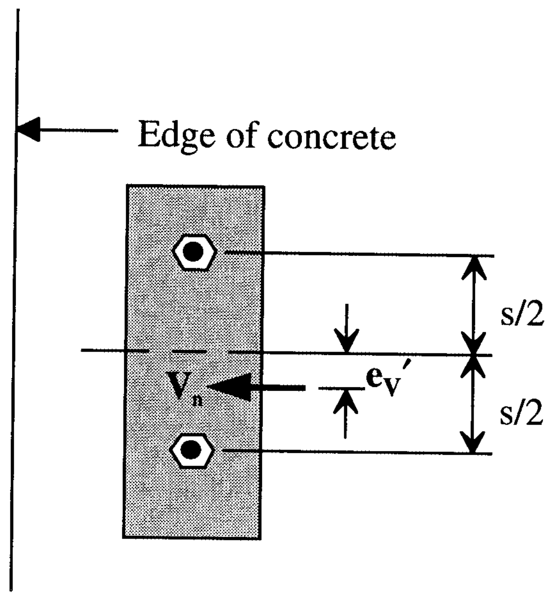
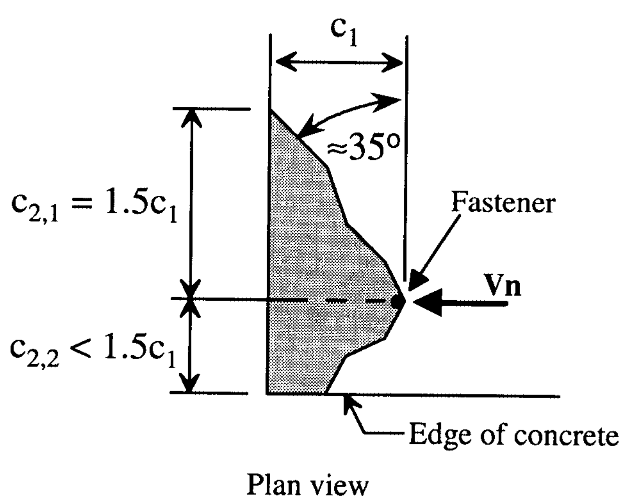


Figure B.6.2(e) Definition of dimension e_V¢



For calculation of Ψ_6 use smaller of $c_{2,1}$ and $c_{2,2}$ for c_2
(In case shown $c_2 = c_{2,2}$)

Figure B.6.2(f) Dimension c_2 for edge proximity modification factor

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B.7 — Interaction of tensile and shear forces

The shear-tension interaction expression has traditionally been expressed as:

$$\left(\frac{N}{N_n}\right)^{\alpha} + \left(\frac{V}{V_n}\right)^{\alpha} \leq 1.0$$

where α varies from 1 to 2. The current tri-linear recommendation is a simplification of the expression where $\alpha = 5/3$ (Fig. B.7). The limits were chosen to eliminate the requirement for computation of interaction effects where very small values of the second force are present. However, any other interaction expression that is verified by test data can be used under A.4.3.

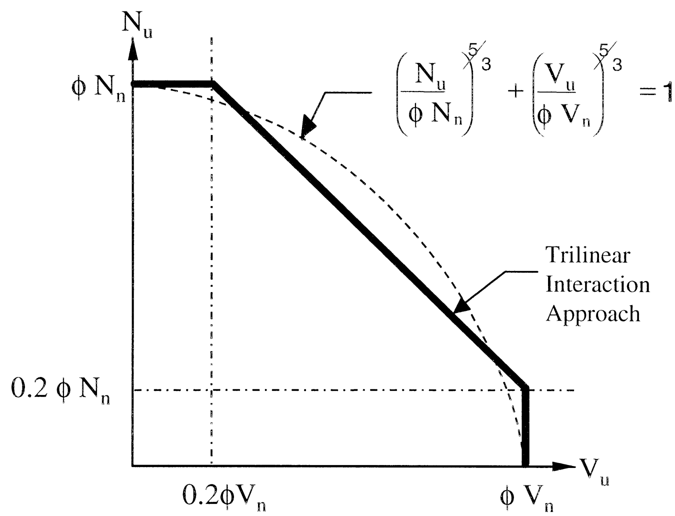

Figure B.7 Shear and tensile load interaction equation

B.8 — Minimum spacings and edge distances

The minimum spacings and edge distances are very dependent on the fastener characteristics. Splitting of the surrounding concrete can be produced in subsequent torquing during connection of attachments to cast-in fasteners.

B.8.1 — Since the edge cover over a deep embedment close to the edge can have a significant effect on the side-face blowout strength of A.5.4, in addition to the normal concrete cover requirements, the designer may wish to use larger cover to increase the side-face blowout strength.

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Appendix C - Design Examples

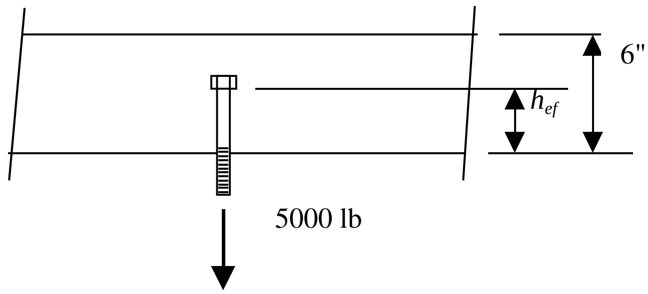
This appendix includes the following design examples:

Example 1 – Single Tension Fastener Away from Edges
Example 2 – Group of Tension Fasteners Near an Edge
Example 3 – Single Fastener in Shear Near an Edge
Example 4 – Single Fastener in Tension and Shear Near an Edge
Example 5 – Group of Tension and Shear Fasteners Near Two Edges
Example 6 – Group of Tension Fasteners Near an Edge with Eccentricity

Example 1 - Single Tension Fastener Away from Edges

Design a single headed bolt installed in the bottom of a 6 in. slab to support a 5000 lb service dead load.

$f_c^{'} = 4000 \text{ psi}$



Calculations and Discussion	Code Reference
1. Determine factored design load	*ACI 318 9.2*
$N_u = 1.4\ (5{,}000) = 7{,}000$ lb	
2. Determine fastener diameter and material	*A.5.1*
The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength. As a result, it is usually economical to design the fastener using a mild steel rather than a high strength steel. ASTM A307 is the best choice. ASTM A307 bolts and studs (i.e., un-headed bar stock with each end threaded for nuts) are available in three grades. If a grade is not specified, Grade A will be furnished.	
Assume an ASTM A307 Grade A bolt for this example.	
The basic requirement for the fastener steel is:	
$\phi N_s \geq N_u$	*Eq. (A-1)*
where:	
$\phi = 0.90$	*A.4.4*
Per the ***Ductile Steel Element*** definition in A.1, ASTM A307 steel qualifies as a ductile steel element (18% minimum elongation in 2 in., see Table 1). Per ASTM A307, there is no minimum yield strength for Grade A, only the minimum tensile strength is provided, therefore *Eq. (A-3)* is not applicable.	
$N_s = n\, A_{se}\, (0.8\, f_{ut})$	*Eq. (A-4)*

For design purposes, *Eq. (A-1)* with *Eq. (A-4)* may be rearranged as:

$$A_{se} \geq \frac{N_u}{\phi\, n(0.8\, f_{ut})}$$

Substituting:

$$A_{se} = \frac{7000}{0.9(1)(0.8(60,000))} = 0.162 \text{ in.}^2$$

Per Table 2, a 5/8 in. diameter threaded fastener will satisfy this requirement (A_{se} = 0.226 in.^{2}).

3. Determine the required embedment length (h_{ef}) *A.5.2*

The basic requirement for the single fastener embedment is:

$$\phi\, N_{cb} \geq N_u$$ *Eq. (A-1)*

where:

$$\phi = 0.75$$ *A.4.4*

No supplementary reinforcement has been provided (i.e., hairpin type reinforcement surrounding the fastener and anchored into the concrete).

$$N_{cb} = \frac{A_N}{A_{No}} \psi_2 \psi_3 N_b$$ *Eq. (A-5a)*

where:

$\frac{A_N}{A_{No}}$ and ψ_2 terms are 1.0 for single fasteners away from edges

$$N_b = 24\sqrt{f_c'}\, h_{ef}^{1.5}$$ *Eq. (A-7a)*

For design of a single fastener away from edges, *Eq. (A-1)* with *Eq. (A-5a)* and *Eq. (A-7a)* may be rearranged as:

$$h_{ef} = \left(\frac{N_u}{\phi\, \psi_3\, 24\sqrt{f_c'}} \right)^{\frac{2}{3}}$$

where:

ψ_3 = 1.0 for locations where concrete cracking is likely to occur (i.e., the bottom of the slab)

Substituting:

$$h_{ef} = \left(\frac{7000}{0.75(1.0)24\sqrt{4000}}\right)^{\frac{2}{3}} = 3.36 \text{ in.}$$

specify 3-1/2 in. embedment for this fastener

Note: The case of a single fastener away from an edge is essentially the only case where h_{ef} can be solved for directly. Whenever edges or adjacent fasteners are present, the solution for h_{ef} is iterative.

4. Determine head size for fastener — *A.5.3*

The basic requirement for pullout strength (i.e., the strength of the fastener related to the embedded head of the fastener having insufficient bearing area so that the fastener pulls out without a full depth concrete breakout failure) is:

$$\phi N_{pn} \geq N_u$$ *Eq. (A-1)*

where:

$\phi = 0.75$ *A.4.4*

No supplementary reinforcement has been provided.

$$N_{pn} = \psi_4 N_p$$ *Eq. (A-10)*

where:

$$N_p = A_b 8 f_c'$$ *Eq. (A-11a)*

ψ_4 = 1.0 for locations where concrete cracking is likely to occur (i.e., the bottom of the slab)

For design purposes *Eq. (A-1)* with *Eq. (A-10)* and *Eq. (A-11a)* may be rearranged as:

$$A_b = \frac{N_u}{\phi \psi_4 8 f_c'}$$

Substituting:

$$A_b = \frac{7000}{0.75(1.0)(8)(4000)} = 0.292 \text{ in.}^2$$

As shown in Table 2, any type of standard head (square, heavy square, hex, or heavy hex) is acceptable for this 5/8 in. diameter fastener.

A.5.4

5. Evaluate Side-Face Blowout

Since this fastener is located far from a free edge of concrete ($c > 0.4\ h_{ef}$) this type of failure mode is not applicable. *A.8*

6. Required Edge Distances and Spacings to Preclude Splitting Failure

Since this fastener is located far from a free edge of concrete, this type of failure mode is not applicable.

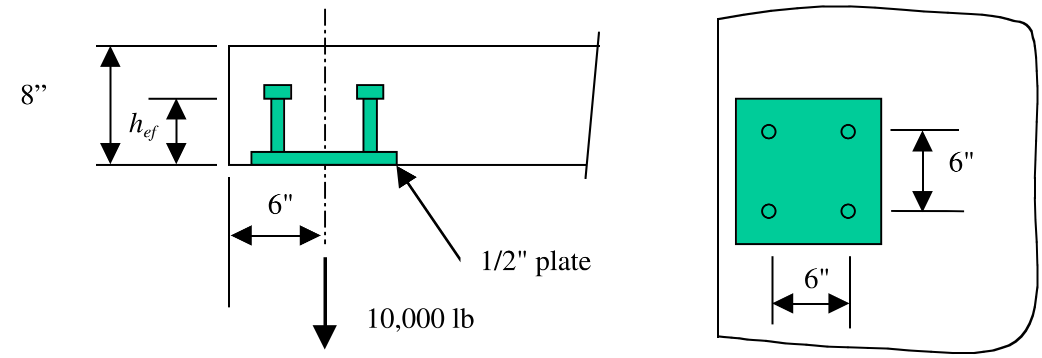
7. Summary:

Use an ASTM A307 5/8 in. diameter bolt with a 3-1/2 in. embedment.

Example 2 - Group of Tension Fasteners Near an Edge

Design a group of four welded, headed studs spaced 6 in. on center and loaded with a 10,000 lb service dead load from suspended mechanical equipment at the edge of an HVAC chase. The fastener group is to be installed in the bottom of an 8 in. thick slab with the centerline of the connection 6 in. from a free edge of the slab.

$f_c' = 4000$ psi



Calculations and Discussion	Code Reference
1. Determine factored design load	*ACI 318 9.2*
$N_u = 1.4\ (10{,}000) = 14{,}000$ lb	
2. Determine fastener diameter	*A.5.1*
Assume AWS D1.1 Type B welded, headed studs.	
The basic requirement for the fastener steel is:	
$\phi N_s \geq N_u$	*Eq. (A-1)*
where:	
$\phi = 0.90$	*A.4.4*
Per the ***Ductile Steel Element*** definition in A.1, AWS D1.1 Type B studs qualify as a ductile steel element (20% minimum elongation in 2 in., see Table 1).	*A.2.1*
$N_s = n\, A_{se}\, (0.8\, f_{ut})$	*Eq. (A-4)*
Per AWS D1.1, the minimum yield strength for Type B studs is determined by the 0.2% offset method (see Table 1). This indicates that this type of steel does not	

have a well-defined yield point, therefore, *Eq. (A-3)* is not applicable.

For design purposes, *Eq. (A-1)* with *Eq. (A-4)* may be rearranged as:

$$A_{se} = \frac{N_u}{\phi n (0.8 f_{ut})}$$

Substituting:

$$A_{se} = \frac{14,000}{0.9(4)(0.8(60,000))} = 0.081 \text{ in.}^2$$

Per Table 2, 1/2 in. diameter welded, headed studs will satisfy this requirement (A_{se} = 0.196 in.^{2}).

Note: Per AWS D1.1 Table 7.1, Type B welded studs are 1/2 in., 5/8 in., 3/4 in., 7/8 in., and 1 in. diameters. Although individual manufacturers may list smaller diameters they are not explicitly covered by AWS D1.1

3. Determine the required embedment length (h_{ef}) *A.5.2*

The basic requirement for embedment of a group of fasteners is:

$$\phi N_{cbg} \geq N_u$$ *Eq. (A-1)*

where:

$$\phi = 0.75$$ *A.4.4*

Condition B applies since no supplementary reinforcement has been provided (i.e., hairpin type reinforcement surrounding the fastener and anchored into the concrete).

$$N_{cbg} = \frac{A_N}{A_{No}} \psi_1 \psi_2 \psi_3 N_b$$ *Eq. (A-5b)*

Since this fastening is likely to be affected by both group effects and edge effects, the embedment length h_{ef} cannot be solved for directly. In this case, an embedment length must be assumed at the outset and then be proven to satisfy the requirement of *Eq. (A-1)*.

Note: Welded studs are generally available in fixed lengths. Available lengths may be determined from manufacturers' catalogs. For example, the Nelson Stud web page (www.trw.com/nelsonstud) indicates an effective embedment of 4 in. for a standard 1/2 in. concrete anchor stud.

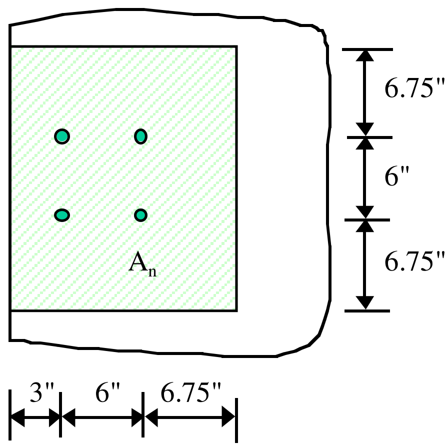
Assume an effective embedment length of $h_{ef} = 4.5$ in.

Note: The effective embedment length for the welded stud fastening is the effective embedment length of the stud plus the thickness of the embedded plate.

Evaluate the terms in *Eq. (A-5b)* with $h_{ef} = 4.5$ in. *A.5.2.1*

Determine A_N and A_{No} for the fastening:

A_N is the projected area of the failure surface as approximated by a rectangle with edges bounded by 1.5 h_{ef} (1.5 x 4.5 = 6.75 in. in this case) and free edges of the concrete from the centerlines of the fasteners.



$$A_N = (3+6+6.75)(6.75+6+6.75) = 307 \text{ in.}^2$$ *Eq. (A-6)*

$$A_{No} = 9\,h_{ef}^2 = 9\,(4.5)^2 = 182 \text{ in.}^2$$ *A.5.2.4*

Determine ψ_1 for the fastening:

$\psi_1 = 1.0$ (no eccentricity in the connection) *A.5.2.5*

Determine ψ_2 for the fastening:

Eq. (A-9b)

$$\psi_2 = 0.7 + 0.3\frac{c_{min}}{1.5\,h_{ef}}$$

$$\psi_2 = 0.7 + 0.3\frac{3.0}{1.5(4.5)} = 0.83$$ A.5.2.6

Determine ψ_3 for the fastening:

ψ_3 = 1.0 for locations where concrete cracking is likely to occur (i.e., the bottom of the slab)

A.5.2.2

Determine N_b for the fastening:

Eq. (A-7a)

$$N_b = 24\sqrt{f_c'}\,h_{ef}^{1.5} = 24\sqrt{4000}\,(4.5)^{1.5} = 14,490 \text{ lb}$$

Substituting into *Eq. (A-5b)*:

$$N_{cbg} = \left[\frac{307}{182}\right](1.0)(0.83)(1.0)(14,490) = 20,287 \text{ lb}$$

The final check on the assumption of h_{ef} = 4.5 in. is satisfied by meeting the requirements of *Eq. (A-1)*:

(0.75) (20,287) ≥ 14,000

15,215 > 14,000 OK

Specify a 4 in. length for the welded, headed studs with the 1/2 in. thick base plate. A.5.3

Eq. (A-1)

4. Determine if welded stud head size is adequate

$\phi N_{pn} \geq N_u$

A.4.4

where:

$\phi = 0.75$

Eq. (A-10)

No supplementary reinforcement has been provided.

$$N_{pn} = \psi_4 N_p$$

Eq. (A-11a)

where:

$$N_p = A_b 8 f_c'$$

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ψ_4 = 1.0 for locations where concrete cracking is likely to occur (i.e., the bottom of the slab)

For design purposes *Eq. (A-1)* with *Eq. (A-10)* and *Eq. (A-11a)* may be rearranged as:

$$A_b = \frac{N_u}{\phi \psi_4 8 f_c'}$$

For the group of four studs the individual factored tension load N_u on each stud is:

$$N_u = \frac{14,000}{4} = 3500\ lb$$

Substituting:

$$A_b = \frac{3500}{0.75(1.0)(8)(4000)} = 0.146\ \text{in.}^2$$

The bearing area of welded, headed studs should be determined from manufacturers' catalogs. As shown on the Nelson Stud web page the diameter of the head for a 1/2 in. diameter stud is 1 in.

A.5.4

$$A_{b,provided} = \frac{\pi}{4}(1.0^2 - 0.5^2) = 0.589\ \text{in}^2 > 0.146\ \text{in.}^2 \quad \text{OK}$$

5. Evaluate Side-Face Blowout

Side-face blowout needs to be considered when the edge distance from the centerline of the fastener to the nearest free edge is less than 0.4 h_{ef}. For this example:

0.4 h_{ef} = 0.4 (4.5) = 1.8 in. < 3 in. actual edge distance OK

A.8

The side-face blowout failure mode is not applicable.

6. Required Edge Distances and Spacings to Preclude Splitting Failure

Since a welded, headed fastener is not torqued the minimum cover requirements of ACI 318 Section 7.7 apply.

Per ACI 318 Section 7.7 the minimum clear cover for a 1/2 in. bar is 3/4 in. which is less than the 2-3/4 in. provided (3 - 1/4 = 2-3/4 in.) - OK

7. Summary:

   Use ASW D1.1 Type B 1/2 in. diameter welded studs with an effective 4.5 in. embedment (4" from the stud plus 1/2 in. from the embedded plate).

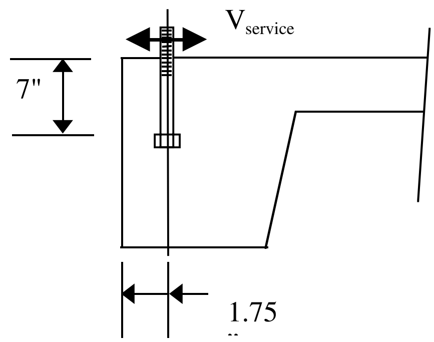
Example 3 - Single Fastener in Shear Near an Edge

Determine the reversible service wind load shear capacity for a single 1/2 in. diameter headed bolt with a 7 in. embedment installed with its centerline 1-3/4 in. from the edge of a concrete foundation.

Note: This is the minimum anchorage requirement at the foundation required by IBC 2000 Section 2308.6 for conventional light-frame wood construction. The 1-3/4 in. edge distance represents a typical connection at the base of wood framed walls using 2x4 members.

$f_c' = 4000$ psi

ASTM A307 Grade A bolt



Calculations and Discussion	Code Reference

1. This problem provides the anchor diameter, embedment length, and material and requires computing the maximum service shear load from wind load. In this case, it is best to first determine the controlling factored shear load V_u based on the smaller of the steel strength and embedment strength then as a last step determine the maximum service load. Step 6 of this example provides the conversion of the controlling factored shear load V_u to a service load due to wind.

2. Determine V_u as controlled by the fastener steel *A.6.1*

   $\phi V_s \geq V_u$ *Eq. (A-2)*

   where:

   $\phi = 0.90$ *A.4.4*

   Per the ***Ductile Steel Element*** definition in A.1, ASTM A307 steel qualifies as a ductile steel element (18% minimum elongation in 2 in., see Table 1). *A.2.1*

   $V_s = n\ 0.6\ A_{se}\ f_{ut}$ *Eq. (A-15)*

   Per ASTM A307, there is no minimum yield strength for Grade A, only the minimum tensile strength is provided, therefore *Eq. (A-14)* is not applicable.

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A_{se} for the 1/2 in. threaded bolt is 0.142 in.^{2} (Table 2)

f_{ut} for the ASTM A307 Grade A bolt is 60,000 psi (Table 1)

To determine V_u for the steel strength *Eq. (A-2)* can be combined with *Eq. (A-15)* to give:

$$V_u = \phi V_s = \phi n (0.6 A_{se}) f_{ut}$$

Substituting, V_u as controlled by steel strength is:

$$V_u = \phi V_s = 0.9(1)(0.6(0.142))(60,000) = 4,600 \text{ lb}$$

3. Determine V_u for embedment strength governed by concrete breakout strength with shear directed toward a free edge *A.6.2*

$\phi V_{cb} \geq V_u$ *Eq. (A-2)*

where:

$\phi = 0.75$ *A.4.4*

No supplementary reinforcement has been provided (i.e., hairpin type reinforcement surrounding the fastener and anchored into the concrete).

$$V_{cb} = \frac{A_v}{A_{vo}} \psi_6 \psi_7 V_b$$ *Eq. (A-18a)*

where:

$\frac{A_v}{A_{vo}}$ and ψ_6 terms are 1.0 for single shear fasteners not influenced by more than one free edge

ψ_7 = 1.0 for locations where concrete cracking is likely to occur (i.e., the edge of the foundation is susceptible to shrinkage cracks)

$$V_b = 7\left(\frac{\ell}{d_0}\right)^{0.2} \sqrt{d_0} \sqrt{f_c'} c_1^{1.5}$$

where:

ℓ = load bearing length of the anchor for shear, not to exceed $8d_0$ — A.2.1

For this problem $8d_o$ will control:

$\ell = 8d_o = 8\ (0.5) = 4.0$ in.

To determine V_u for the embedment strength governed by concrete breakout strength *Eq. (A-2)* can be combined with *Eq. (A-16a)* and *Eq. (A-18a)* to give:

$$V_u = \phi V_{cb} = \phi 7 \left(\frac{\ell}{d_0}\right)^{0.2} \sqrt{d_0}\ \sqrt{f_c'}\ c_1^{1.5}$$

Substituting, V_u for the embedment strength as controlled by concrete breakout strength is:

$$V_u = \phi V_{cb} = 0.75(7)\left(\frac{8(0.5)}{(0.5)}\right)^{0.2} \sqrt{0.5}\ \sqrt{4000}\ 1.75^{1.5} = 824 \text{ lb}$$

4. Determine V_u for embedment strength governed by concrete pryout strength — A.6.3

Note: The pryout failure mode is normally only a concern for shallow, stiff fasteners. Since this example problem addresses both shear directed toward the free edge and shear directed inward from the free edge, the pryout strength will be evaluated.

$\phi V_{cp} \geq V_u$ — Eq. (A-2)

where:

$\phi = 0.75$

No supplementary reinforcement has been provided.

$V_{cp} = k_{cp} N_{cb}$ — Eq. (A-21)

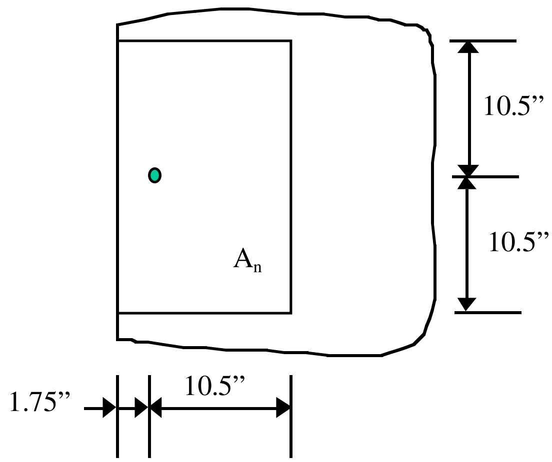
where:

$k_{cp} = 2.0$ for $h_{ef} > 2.5$ in.

$$N_{cb} = \frac{A_N}{A_{No}} \psi_2 \psi_3 N_b$$ Eq. (A-5a)

Evaluate the terms of *Eq. (A-5a)* for this problem:

A_N is the projected area of the tensile failure surface as approximated by a rectangle with edges bounded by 1.5 h_{ef} (1.5 x 7 = 10.5 in. in this case) and free edges of the concrete from the centerline of the fastener.



$$A_N = (1.75 + 10.5)(10.5 + 10.5) = 257 \text{ in}^2$$

$A_{No} = 9\ h_{ef}^2 = 9\ (7.0)^2 = 441$ in.^{2} *Eq. (A-6)*

Determine ψ_2 for the fastening: *A.5.2.5*

$$\psi_2 = 0.7 + 0.3\frac{c_{min}}{1.5h_{ef}}$$ *Eq. (A-9b)*

$$\psi_2 = 0.7 + 0.3\frac{1.75}{1.5(7.0)} = 0.75$$

Determine ψ_3 for the fastening: *A.5.2.6*

ψ_3 = 1.0 for locations where concrete cracking is likely to occur (i.e., the edge of the foundation is susceptible to shrinkage cracks)

Determine Nb for the fastening: *A.5.2.2*

$$N_b = 24\sqrt{f_c'}\,h_{ef}^{1.5} = 24\sqrt{4000}\,(7.0)^{1.5} = 28{,}112 \text{ lb}$$ *Eq. (A-7a)*

Substituting into Eq. (A-5a):

$$N_{cb} = \left[\frac{257}{441}\right](1.0)(0.75)(1.0)(28{,}112) = 12{,}287 \text{ lb}$$

To determine Vu for the embedment strength governed by pryout strength Eq. (A-2) can be combined with Eq. (A-21) to give:

$$V_u = \phi V_{cp} = \phi k_{cp} N_{cb}$$

Substituting, V_u for the embedment strength governed by pryout is:

$$V_u = \phi V_{cp} = 0.75\ (2.0)(12{,}287) = 18{,}430 \text{ lb}$$

5. Required Edge Distances and Spacings to Preclude Splitting Failure

Since a headed fastener used to attach wood frame construction is not likely to be torqued significantly, the minimum cover requirements of ACI 318 Section 7.7 apply. *A.8*

Per ACI 318 Section 7.7 the minimum clear cover for a 1/2 in. bar is 1-1/2 in. when exposed to earth or weather. The clear cover provided for the bolt is exactly 1-1/2 in. (1-3/4 in. to bolt centerline less one half bolt diameter). Note that the bolt head will have slightly less cover (1-3/16 in. for a hex head) - say OK *ACI 318 7.7*

6. Summary:

The factored shear load ($V_u = \phi V_n$) based on steel strength and embedment strength (concrete breakout and pryout) can be summarized as:

Steel strength, (ϕV_s):	4,600 lb
Embedment strength - concrete breakout, (ϕV_{cb}):	824 lb ← controls
Embedment strength - pryout, (ϕV_{cp}):	18,430 lb

In accordance with ACI 318 Section 9.2 the load factor for wind load is 1.3:

$$V_{service} = \frac{V_u}{1.3} = \frac{824}{1.3} = 634 \text{ lb}$$

ACI 318 9.2

The reversible service load shear strength from wind load of the IBC 2000 Section 2308.6 minimum foundation connection for conventional wood-frame construction (1/2 in. diameter bolt embedded 7 in.) is 634 lb per bolt.

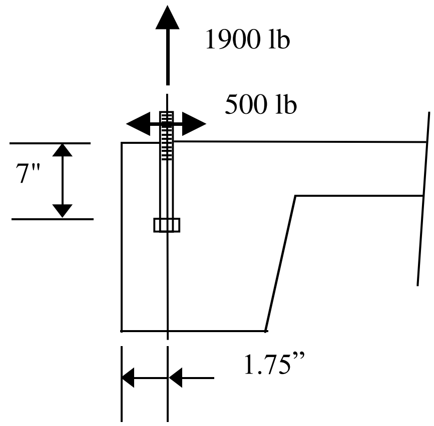
Example 4 - Single Fastener in Tension and Shear Near an Edge

Determine if a single 1/2 in. diameter hex headed bolt with a 7 in. embedment installed with its centerline 1-3/4 in. from the edge of a concrete foundation is adequate for a service tension load from wind of 1900 lb and reversible service shear load from wind of 500 lb.

Note: This is an extension of Example 3 that includes a tension load on the fastener as well as a shear load.

f'_c = 4000 psi

ASTM A307 Grade A hex head bolt



Calculations and Discussion	Code Reference

1. Determine the factored design loads — *ACI 318 9.2*

   N_u = 1.3 (1900) = 2470 lb

   V_u = 1.3 (500) = 650 lb

2. This is a tension/shear interaction problem where values for both the design tensile strength (ϕN_n) and design shear strength (ϕV_n) will need to be determined. ϕN_n is the smallest of the design tensile strengths as controlled by steel (ϕN_s), concrete breakout (ϕN_{cb}), pullout (ϕN_{pn}), and side-face blowout (ϕN_{sb}). ϕV_n is the smallest of the design shear strengths as controlled by steel (ϕV_s), concrete breakout (ϕV_{cb}), and pryout (ϕV_{cp}). — *A.7*

3. Determine the design tensile strength (ϕN_n) — *A.5*

   Steel strength, (ϕN_s): — *A.5.1*

   $\phi N_s = \phi\, n\, A_{se}\, (0.8\, f_{ut})$ — *Eq. (A-4)*

   Note: Eq. (A-4) is appropriate for an ASTM A307 Grade A bolt since this material does not have a specified minimum yield strength (see Table 1). — *A.5.1.2*

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where:

$\phi = 0.90$

A.4.4

Per Table 1, the ASTM A307 Grade A bolt meets the ***Ductile Steel Element*** definition of Section A.1.

$A_{se} = 0.142$ in.^{2} (see Table 2)

$f_{ut} = 60{,}000$ psi (see Table 1)

Substituting:

$\phi N_s = 0.90\ (1)\ (0.142)\ ((0.8)(60{,}000)) = 6134$ lb

Concrete breakout strength (ϕN_{cb}):

A.5.2

Since no supplementary reinforcement has been provided, $\phi = 0.75$

A.4.4

In the process of calculating the pryout strength for this fastener in Example 3 Step 4, N_{cb} for this fastener was found to be 12,287 lb

$\phi N_{cb} = 0.75\ (12{,}287) = 9215$ lb

Pullout strength (ϕN_{pn})

A.5.3

$\phi N_{pn} = \phi\ \psi_4\ N_p$

Eq. (A-10)

where:

$\phi = 0.75$, no supplementary reinforcement has been provided

A.4.4

$\psi_4 = 1.0$, cracking may occur at the edges of the foundation

$N_p = A_b 8 f_c'$ A.5.3.5

$A_b = 0.291$ in.^{2}, for 1/2 in. hex head bolt (see Table 2) Eq. (A-11a)

Pullout Strength (ϕN_{pn})

$\phi N_{pn} = 0.75\ (1.0)\ (0.291)\ (8)\ (4000) = 6984$ lb

Concrete side-face blowout strength (ϕN_{sb}) *A.5.4*

The side-face blowout failure mode must be investigated when the edge distance (c) is less than 0.4 h_{ef} *A.5.4.1*

0.4 h_{ef} = 0.4 (7) = 2.80 in. > 1.75 in.

Therefore, the side-face blowout strength must be determined

$$\phi N_{sb} = \phi \left(160\, c \sqrt{A_b}\, \sqrt{f'_c}\right)$$

Eq. (A-12)

where:

ϕ = 0.75, no supplementary reinforcement has been provided *A.4.4*

c = 1.75 in.

A_b = 0.291 in.^{2}, for 1/2 in. hex head bolt (see Table 2)

Substituting:

$$\phi N_{sb} = 0.75\left(160\,(1.75)\sqrt{0.291}\,\sqrt{4000}\right) = 7165\,\text{lb}$$

Summary of steel strength, concrete breakout strength, pullout strength, and side-face blowout strength for tension:

Steel strength, (ϕN_s):	6134 lb ← controls	*A.5.1*
Embedment strength - concrete breakout, (ϕN_{cb}):	9215 lb	*A.5.2*
Embedment strength - pullout, (ϕN_{pn}):	6984 lb	*A.5.3*
Embedment strength - side-face blowout, (ϕN_{sb}):	7165 lb	*A.5.4*

Therefore:

ϕN_n = 6,134 lb

4. Determine the design shear strength (ϕV_n)

Summary of steel strength, concrete breakout strength, and pryout strength for shear from Example 3: *A.6*

Steel strength, (ϕV_s):	4600 lb	*A.6.1*
Embedment strength - concrete breakout, (ϕV_{cb}):	824 lb ← controls	*A.6.2*
Embedment strength - pryout, (ϕV_{cp}):	18,430 lb	*A.6.3*

Therefore:

$\phi V_n = 824$ lb

5. Check Tension and Shear Interaction *A.7*

If $V_u \leq 0.2\phi V_n$ then the full tension design strength is permitted *A.7.1*

$V_u = 650$ lb

$0.2\phi V_n = 0.2 (824) = 165$ lb

V_u exceeds $0.2\phi V_n$, the full tension design strength is not permitted

If $N_u \leq 0.2 \phi N_n$ then the full shear design strength is permitted *A.7.2*

$N_u = 24700$ lb

$0.2\phi N_n = 0.2 (6134) = 1227$ lb

N_u exceeds $0.2\phi N_n$, the full shear design strength is not permitted

A.7.3

The interaction equation must be used

Eq. (A-22)

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \leq 1.2$$

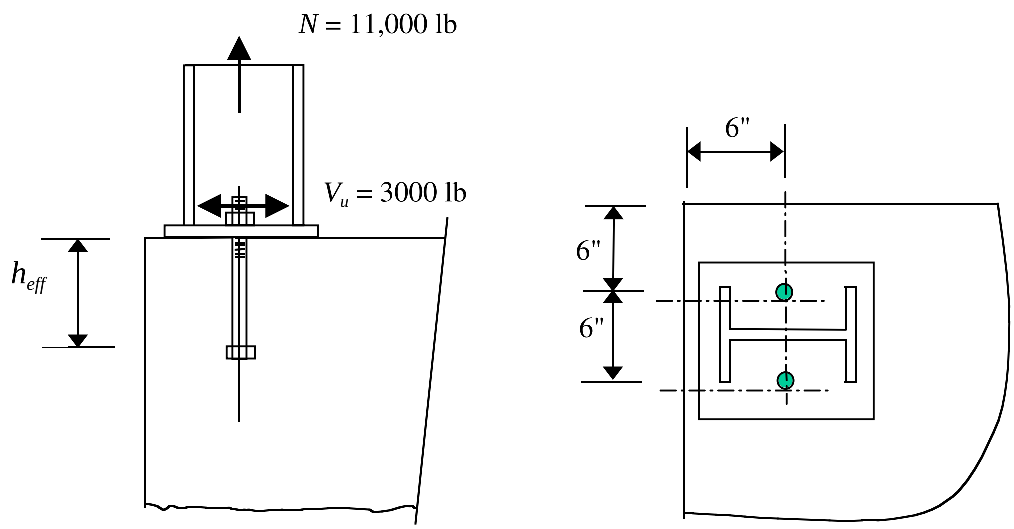
$$\frac{2470}{6134} + \frac{1.19}{824} = 1.19 < 1.2 \quad \text{OK}$$

Use a 1/2 in. diameter ASTM A307 Grade A hex headed bolt embedded 7 in.

Example 5 - Group of Tension and Shear Fasteners Near Two Edges

Design a group of two headed bolts spaced as shown to support a 11,000 lb factored tension load and 3000 lb reversible factored shear load resulting from wind load. The connection is located at the base of a pinned end column in a corner of the building foundation.

$f_c' = 4000$ psi



Calculations and Discussion	Code Reference

1. The solution to this example is found by assuming the size of the fastener, then checking compliance with the design provisions. Try two 5/8 in. ASTM A307 Grade A hex headed bolts with $h_{ef} = 8$ in.

2. As in Example 4, this is a tension/shear interaction problem where values for both the design tensile strength (ϕN_n) and design shear strength (ϕV_n) will need to be determined. ϕN_n is the smallest of the design tensile strengths as controlled by steel (ϕN_s), concrete breakout (ϕN_{cb}), pullout (ϕN_{pn}), and side-face blowout (ϕN_{sb}). ϕV_n is the smallest of the design shear strengths as controlled by steel (ϕV_s), concrete breakout (ϕV_{cb}), and pryout (ϕV_{cp}). *A.7*

3. Determine the design tensile strength (ϕN_n)

A.5

Steel strength, (ϕN_s):

A.5.1

$$\phi N_s = \phi\, n\, A_{se}\, (0.8\, f_{ut})$$

Eq. (A-4)

Note: Eq. (A-4) is appropriate for an ASTM A307 Grade A bolt since this material does not have a specified minimum yield strength (see Table 1).

where:

$\phi = 0.90$

A.4.4

Per Table 1, the ASTM A307 Grade A bolt meets the ***Ductile Steel Element*** definition of Section A.1.

$A_{se} = 0.226$ in.^{2} (see Table 2)

$f_{ut} = 60{,}000$ psi (see Table 1)

Substituting:

$\phi N_s = 0.90\ (2)\ (0.226)\ ((0.8)(60{,}000)) = 19{,}526$ lb

Concrete breakout strength (ϕN_{cbg}):

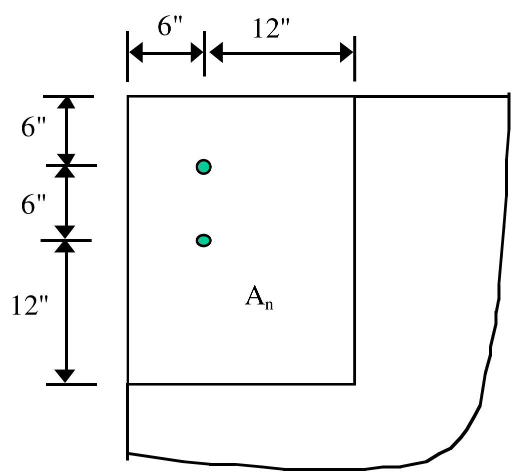
$$\phi N_{cbg} = \phi \frac{A_n}{A_{no}} \psi_1 \psi_2 \psi_3 N_b$$ A.5.2

Eq. (A-5b)

Since no supplementary reinforcement has been provided, $\phi = 0.75$

Determine A_N and A_{No} for the fastening: A.4.4

A_N is the projected area of the failure surface as approximated by a rectangle with edges bounded by 1.5 h_{ef} (1.5 x 8.0 = 12.0 in. in this case) and free edges of the concrete from the centerlines of the fasteners. A.5.2.1



$A_N = (6 + 12)(6 + 6 + 12) = 432$ in.^{2}

$A_{No} = 9\ h_{ef}^{\ 2} = 9\ (8)^2 = 576$ in.^{2}

Determine ψ_1 for the fastening:

ψ_1 = 1.0 (no eccentricity in the connection) — *Eq. (A-6)*

Determine ψ_2 for the fastening: — *A.5.2.4*

$$\psi_2 = 0.7 + 0.3\frac{c_{min}}{1.5h_{ef}}$$

A.5.2.5

Eq. (A-9b)

$$\psi_2 = 0.7 + 0.3\frac{6.0}{1.5(8.0)} = 0.85$$

Determine ψ_3 for the fastening:

A.5.2.6

ψ_3 = 1.0 for locations where concrete cracking is likely to occur (i.e., the edge of the foundation)

Determine N_b for the fastening:

A.5.2.2

$$N_b = 24\sqrt{f_c'}\,h_{ef}^{1.5} = 24\sqrt{4000}\,(8.0)^{1.5} = 34,346 \text{ lb}$$

Eq. (A-7a)

Substituting into *Eq. (A-5b)*:

$$\phi N_{cbg} = 0.75\left[\frac{432}{576}\right](1.0)(0.85)(1.0)(34,346) = 16,422 \text{ lb}$$

Pullout strength (ϕN_{pn})

$\phi N_{pn} = \phi\, \psi_4\, N_p$ — *A.5.3*

where: — *Eq. (A-10)*

$\phi = 0.75$, no supplementary reinforcement has been provided

A.4.4

ψ_4 = 1.0, cracking may occur at the edges of the foundation

A.5.3.5

$$N_p = A_b 8 f_c'$$

Eq. (A-11a)

A_b = 0.454 in.^{2}, for 5/8 in. hex head bolt (see Table 2)

Substituting into *Eq. (A-10)* with 2 bolts (ϕN_{pn})

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$\phi N_{pn} = 2\ (0.75\ (1.0)\ (0.454)\ (8)\ (4000)) = 21{,}792$ lb

Concrete side-face blowout strength (ϕN_{sb}) *A.5.4*

The side-face blowout failure mode must be investigated when the edge distance (c) is less than 0.4 h_{ef} *A.5.4.1*

0.4 h_{ef} = 0.4 (8) = 3.2 in. < 6.0 in.

Therefore, the side-face blowout strength is not applicable (N/A)

Summary of steel strength, concrete breakout strength, pullout strength, and side-face blowout strength for tension:

Steel strength, (ϕN_s):	19,526 lb	*A.5.1*
Embedment strength - concrete breakout, (ϕN_{cb}):	16,422 lb ← controls	*A.5.2*
Embedment strength - pullout, (ϕN_{pn}):	21,792 lb	*A.5.3*
Embedment strength - side-face blowout, (ϕN_{sb}):	N/A	*A.5.4*

Therefore:

$\phi N_n = 16{,}422$ lb

4. Determine the design shear strength (ϕV_n) *A.6*

Steel strength, (ϕV_s): *A.6.1*

$\phi V_s = \phi\, n\, 0.6\, A_{se}\, f_{ut}$ *Eq. (A-15)*

Note: Eq. (A-15) is appropriate for an ASTM A307 Grade A bolt since this material does not have a specified minimum yield strength (see Table 1).

where:

A.4.4

$\phi = 0.90$

Per Table 1, the ASTM A307 Grade A bolt meets the ***Ductile Steel Element*** definition of Section A.1.

$A_{se} = 0.226$ in.^{2} (see Table 2)

f_{ut} = 60,000 psi (see Table 1)

Substituting:

ϕV_s = 0.90 (2) (0.6) (0.226) (60,000) = 14,645 lb *A.6.2*

Concrete breakout strength (ϕV_{cbg}): *Eq. (A-16b)*

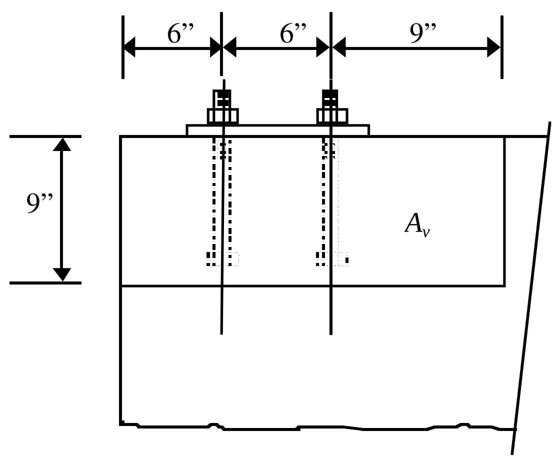
$$\phi V_{cbg} = \phi \frac{A_V}{A_{V0}} \psi_5 \psi_6 \psi_7 V_b$$ *A.4.4*

A.6.2.1

Since no supplementary reinforcement has been provided, $\phi = 0.75$

Determine A_V and A_{V0} for the fastening:

A_V is the projected area of the shear failure surface on the free edge toward which shear is directed. The projected area is approximated by a rectangle with edges bounded by 1.5 c_1 (1.5 x 6.0 = 9.0 in. in this case) and free edges of the concrete from the centerlines of the fasteners and the surface of the concrete. Although the 1.5 c_1 distance is not specified in Section A.6.2.1, it is shown in Appendix B Commentary Figure B.6.2(b).



$$A_V = (6 + 6 + 9)(9) = 189 \text{ in}^2$$

$A_V = 4.5\ c_1^{\,2} = 4.5\ (6)^2 = 162$ in.^{2}

Determine ψ_5 for the fastening: *Eq. (A.17)*

ψ_5 = 1.0 (no eccentricity in the connection) *A.6.2.5*

Determine ψ_6 for the fastening: A.6.2.6

$$\psi_6 = 0.7 + 0.3\frac{c_2}{1.5c_1}$$ Eq. (A-20a)

$$\psi_6 = 0.7 + 0.3\frac{6.0}{1.5(6.0)} = 0.90$$

Determine ψ_7 for the fastening: A.6.2.7

ψ_7 = 1.0 for locations where concrete cracking is likely to occur (i.e., the edge of the foundation)

Determine V_b for the fastening:

$$V_b = 7\left(\frac{\ell}{d_0}\right)^{0.2}\sqrt{d_0}\,\sqrt{f_c'}\,c_1^{1.5}$$ Eq. (A-18a)

where:

ℓ = load bearing length of the anchor for shear, not to exceed $8d_0$ A.0

For this problem $8d_o$ will control:

$\ell = 8d_o = 8\,(0.625) = 5.0$ in. < 8 in. therefore, use $8d_o$

Substituting into *Eq. (A-18a)*:

$$V_b = (7)\left(\frac{8(0.625)}{0.625}\right)^{0.2}\sqrt{0.625}\,\sqrt{4000}\;6.0^{1.5} = 7797\ \text{lb}$$

Substituting into *Eq. (A-16b)*:

$$\phi V_{cbg} = 0.75\left(\frac{189}{162}\right)(1.0)(0.90)(1.0)(7797) = 6140\ \text{lb}$$

Pryout strength ($\phi\, V_{cp}$) A.6.3

Note: The pryout failure mode is normally only a concern for shallow, stiff fasteners. Since this example problem addresses both shear directed toward the free edge and shear directed inward from the free edge, the pryout strength will be evaluated.

$\phi V_{cp} = \phi k_{cp} N_{cb}$ — *Eq. (A-21)*

where:

$\phi = 0.75$, no supplementary reinforcement has been provided — *A.4.4*

$k_{cp} = 2.0$ for $h_{ef} > 2.5$ in.

$$N_{cb} = \frac{A_N}{A_{No}} \psi_2 \psi_3 N_b$$ *Eq. (A-5a)*

$$N_{cb} = \left[\frac{432}{576}\right](0.85)(1.0)(34{,}346) = 21{,}896 \text{ lb}$$

Substituting into *Eq. (A-21)*:

$\phi V_{cp} = 0.75\ (2.0)\ (21{,}896) = 32{,}844$ lb

Summary of steel strength, concrete breakout strength, and pryout strength for shear:

Steel strength, (ϕV_s):	14,645 lb	*A.6.1*
Embedment strength - concrete breakout, (ϕV_{cb}):	6,140 lb ← controls	*A.6.2*
Embedment strength - pryout, (ϕV_{cp}):	32,844 lb	*A.6.3*

Therefore:

$\phi V_n = 6{,}140$ lb

5. Required Edge Distances and Spacings to Preclude Splitting Failure — *A.8*

Since headed cast-in-place fasteners are not likely to be highly torqued, the minimum cover requirements of ACI 318 Section 7.7 apply.

Per ACI 318 Section 7.7 the minimum clear cover for a 5/8 in. bar is 1-1/2 in. when exposed to earth or weather. The clear cover provided for the bolt exceeds this requirement with the 6 in. edge distance to the bolt centerline - OK

6. Check Tension and Shear Interaction — *A.7*

If $V_u \le 0.2\phi V_n$ then the full tension design strength is permitted — *A.7.1*

V_u = 3000 lb

0.2ϕV_n = 0.2 (6140) = 1228 lb

V_u exceeds 0.2ϕV_n, the full tension design strength is not permitted

A.7.2

If $N_u \leq 0.2\ \phi N_n$ then the full shear design strength is permitted

N_u = 11,000 lb

0.2ϕN_n = 0.2 (16,422) = 3284 lb

N_u exceeds 0.2ϕN_n, the full shear design strength is not permitted

The interaction equation must be used *A.7.3*

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \leq 1.2$$ *Eq. (A-22)*

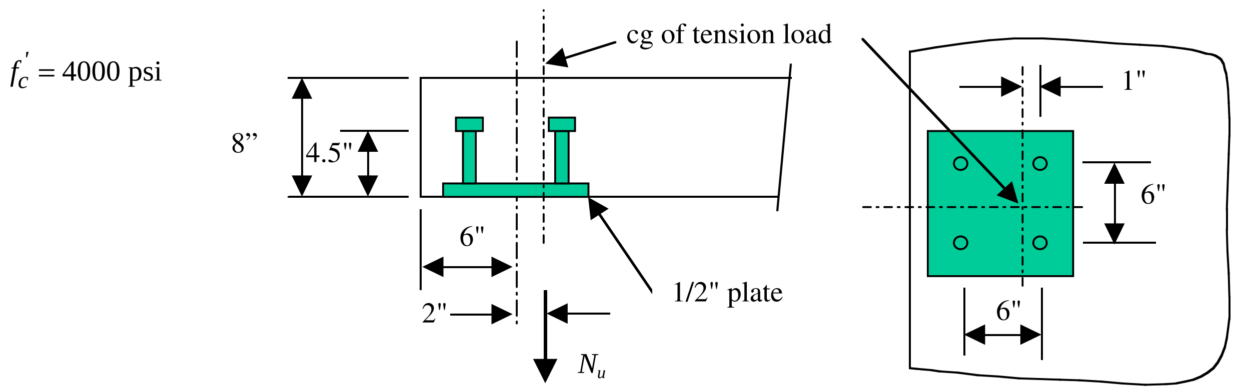
$$\frac{11{,}000}{16{,}422} + \frac{3000}{6140} = 1.16 < 1.2 \text{ OK}$$

Use 5/8 in. diameter ASTM A307 Grade A hex headed bolts embedded 8 in.

Example 6 - Group of Tension Fasteners Near an Edge with Eccentricity

Determine the factored tension load capacity (N_u) for a group of four 1/2 in. x 4 in. AWS D1.1 Type B headed studs spaced 6 in. on center and welded to a 1/2-in.-thick base plate. The centerline of the structural attachment to the base plate has been located 2 in. off of the centerline of the base plate resulting in an eccentricity of the tension load of 2 in. The fastener group is installed in the bottom of an 8 in. slab with the centerline of the connection 6 in. from a free edge of the slab.

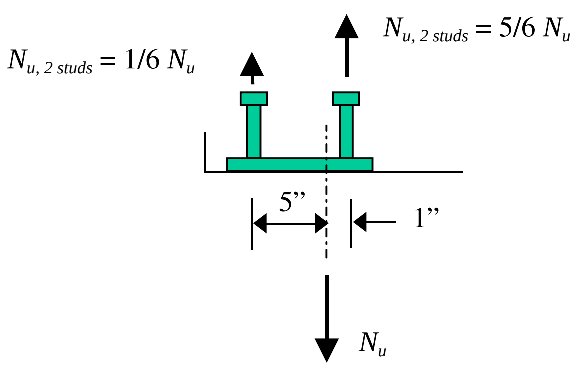
Note: This is the configuration chosen as a solution for Example 2 to support a 14,000 lb factored tension load centered on the connection. The only difference is the eccentricity of the tension load.



Calculations and Discussion	**Code Reference**

1. Determine N_u as controlled by the fastener steel *A.5.1*

   The eccentricity of the tension load will result in a higher force on the interior row of fasteners. Although the studs are welded to the base plate, their flexural stiffness at the joint with the base plate is minimal compared to that of the base plate. Therefore, assume a simple support condition for the base plate:



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Obviously, the interior studs will control the strength related to the fastener steel with $N_{u,\,2\,studs}$ = 5/6 N_u.

Determine N_u for 2 studs:

$N_{u,\,2\,studs} = \phi\, N_s = \phi\, n\, A_{se}\, (0.8\, f_{ut})$ *Eq. (A-4)*

Per AWS D1.1, the minimum yield strength for Type B studs is determined by the 0.2% offset method (see Table 1). This indicates that this type of steel does not have a well-defined yield point, therefore *Eq. (A-3)* is not applicable.

where:

$\phi = 0.90$ *A.4.4*

Per the ***Ductile Steel Element*** definition in Section A.1, AWS D1.1 Type B studs qualify as a ductile steel element (20% minimum elongation in 2 in., see Table 1).

$n = 2$

A_{se} = 0.196 in.^{2} (see Table 2)

f_{ut} = 60,000 (see Table 1)

Substituting:

$N_{u,\,2\,studs} = \phi\, N_s$ = 0.90 (2) (0.196) ((0.8)(60,000) = 16,934 lb

Therefore, N_u as controlled by the fastener steel is:

N_u = 6/5 $N_{u,\,2\,studs}$ = 6/5 (16,934) = 20,321 lb

2. Determine N_u as controlled by concrete breakout

The only difference between concrete breakout strength in this example and Example 2 is the introduction of the eccentricity factor ψ_1. *A.5.2*

From Example 2 with ψ_1 = 1.0:

$$N_u = \phi\, N_{cbg} = \phi \frac{A_n}{A_{no}} \psi_1 \psi_2 \psi_3\, N_b = 15{,}215\ lb$$ *Eq. (A-5b)*

Determine ψ_1 for this example:

$$\psi_1 = \frac{1}{\left(1 + \frac{2e_n'}{3h_{ef}}\right)}$$ *Eq. (A-8)*

where:

e'_n = 2" (distance between centroid of anchor group and tension force)

h_{ef} = 4.5 in. (1/2 in. plate plus 4 in. embedment of headed stud)

Substituting:

$$\psi_1 = \frac{1}{\left(1 + \frac{2(2)}{3(4.5)}\right)} = 0.77$$

Therefore:

N_u = (0.77) (15,215) = 11,716 lb

3. Determine N_u as controlled by pullout strength of a single stud

A.5.3

$$N_{u,1stud} = \phi N_{pn} = \phi \psi_4 N_p = \phi \psi_4 A_b \, 8 \, f_c'$$

where:

$\phi = 0.75$

A.4.4

No supplementary reinforcement has been provided.

ψ_4 = 1.0 for locations where concrete cracking is likely to occur (i.e., the bottom of the slab) *A.5.3.5*

A_b = 1.0 in.^{2} (see Step 4 of Example 2)

Substituting:

$N_{u,\,1\,stud}$ = (0.75) (1.0) (1.0) (8.0) (4,000) = 24,000 lb

Based on fastener steel strength (Step 1) and concrete breakout strength (Step 2), N_u is controlled by concrete breakout strength (N_u = 11,716 lb)

Using the Step 1 relationship of $N_{u,\,2\,studs} = 5/6\ N_u$, the tensile force on one of the interior studs is:

$$N_{u,1stud} = \frac{\frac{5}{6}N_u}{2} = \frac{\frac{5}{6}(11{,}716)}{2} = 4{,}882 \text{ lb} < 24{,}000 \text{ lb OK}$$

Therefore, pullout strength does not control

4. Evaluate Side-Face Blowout A.5.4

Side-face blowout needs to be considered when the edge distance from the centerline of the fastener to the nearest free edge is less than 0.4 h_{ef}. For this example:

0.4 h_{ef} = 0.4 (4.5) = 1.4 in. < 3 in. actual edge distance OK

The side-face blowout failure mode is not applicable.

5. Required Edge Distances and Spacings to Preclude Splitting Failure A.8

Since a welded, headed fastener is not torqued the minimum cover requirements of ACI 318 Section 7.7 apply.

Per ACI 318 Section 7.7 the minimum clear cover for a 1/2 in. bar is 3/4 in. which is less than the 2-3/4 in. provided (3 in. - 1/4 in. = 2-3/4 in.) - OK

6. Summary:

$N_{u,steel}$ = 20,321 lb

$N_{u,\,embedment}$ = 11,716 lb

Embedment capacity based on concrete breakout strength controls

The maximum factored tension load N_u for this fastening is 11,716 lb

Note: Example 2 which resulted in the same connection but without an eccentricity was also controlled by concrete breakout strength but had a factored load capacity of 15,215 lb (see Step 3 of Example 2).

Also from PCA

The following publications may be of interest to readers of this report:

Circular Concrete Tanks without Prestressing (IS072)
Column Shortening in Tall Buildings - Prediction and Compensation (EB108)
Connections for Tilt-Up Wall Construction (EB110)
Design and Control of Concrete Mixtures (EB001)
Design of Concrete Beams for Torsion (EB106)
Design of Concrete Buildings for Earthquake and Wind Forces (EB113)
Design of Concrete Buildings for Earthquake & Wind Forces According to the 1997 Uniform Building Code (EB117)
Design of Low-Rise Concrete Buildings for Earthquake Forces (EB004)
Design of Multistory Reinforced Concrete Buildings for Earthquake Motions (EB032)
Design Provisions for Shearwalls (RD028)
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